```
+))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))),
* CCB Application Notes:                                                *
*                                                                       *
* 1. Character(s) preceded & followed by these symbols (. -) or (+ ,)  *
*    are super- or subscripted, respectively.                           *
*    EXAMPLES:  42m.3-  =  42 cubic meters                              *
*               CO+2,   =  carbon dioxide                               *
*                                                                       *
* 2. All degree symbols have been replaced with the word deg.           *
*                                                                       *
* 3. All plus or minus symbols have been replaced with the symbol +/-. *
*                                                                       *
* 4. All table note letters and numbers have been enclosed in square   *
*    brackets in both the table and below the table.                    *
*                                                                       *
* 5. Whenever possible, mathematical symbols have been replaced with   *
*    their proper name and enclosed in square brackets.                 *
.)))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))-
```

Naval Facilities Engineering Command
200 Stovall Street
Alexandria, Virginia 22322-2300 APPROVED FOR PUBLIC RELEASE
))

Soil Mechanics

DESIGN MANUAL 7.01

REVALIDATED BY CHANGE 1 SEPTEMBER 1986

NAVAL FACILITIES ENGINEERING COMMAND
PUBLICATIONS TRANSMITTAL

SN 0525-LP-300-7056

1. TITLE	NUMBER	DATE
DM-7.01 SOIL MECHANICS	Change 1	September 1986

2. ATTACHMENTS

New Cover
New Record of Document Changes page.
New pages iii through xv.
New pages 7.1-1 and 7.1-2.
New pages 7.1-45 through 7.1-50.
New pages 7.1-115 through 7.1-118.
New pages 7.1-125 through 7.1-126.
New pages 7.1-153 through 7.1-154
New pages 7.1-159 through 7.1-162.
New pages 7.1-175 and 7.1-176.
New pages 7.1-203 and 7.1-204.
New pages 7.1-259 and 7.1-260.
New pages 7.1-283 and 7.1-284.
New page 7.1-307.
New pages 7.1-345 and 7.1-346.
New pages 7.1-A-1 and 7.1-A-2.
New DD Form 1426.

3. EXPLANATION

Remove old cover dated May 1982, and replace with attached new cover dated September 1986.
Insert new Record of Document Changes pages immediately after new cover page.
Remove pages 7.1-iii through 7.1-xviii and replace with new pages iii through xv.
Remove pages 7.1-1 and 7.1-2 and replace with attached new pages 7.1-1 and 7.1-2
Remove pages 7.1-45 through 7.1-50 and replace with attached new pages 7.1-47 through 7.1-50.
Remove pages 7.1-115 through 7.1-118 and replace with attached new pages 7.1-115 through 7.1-118.
Remove pages 7.1-125 through 7.1-126 and replace with attached new pages 7.1-125 through 7.1-126.
Remove pages 7.1-153 and 7.1-154 and replace with attached new pages 7.1-153 and 7.1-154.
Remove pages 7.1-159 through 7.1-162 and replace with attached new pages 7.1-159 through 7.1-162.
Remove pages 7.1-175 and 7.1-176 and replace with attached new pages 7.1-175 and 7.1-176.
Remove pages 7.1-203 and 7.1-204 and replace with attached new pages 7.1-203 and 7.1-204.
Remove pages 7.1-259 and 7.1-260 and replace with attached new pages 7.1-259 and 7.1-260.
Remove pages 7.1-283 and 7.1-284 and replace with attached new pages 7.1-283 and 7.1-284.
Remove page 7.1-307 and replace with attached new page 7.1-307.
Remove pages 7.1-345 and 7.1-346 and replace with attached new pages 7.1-345 and 7.1-346.
Remove pages 7.1-A-1 and 7.1-A-2 and replace with attached new pages 7.1-A-1 and 7.1-A-2.
Insert new DD Form 1426 in back of manual.

DM-7.01 SOIL MECHANICS is revalidated for three years after incorporation of this change.

4. DISTRIBUTION

See reverse.

NAVFAC DM/P MANUAL DISTRIBUTION (01/87):
(1 copy each unless otherwise specified)

SNDL	
39B	COMCB
49	ADMSUPPU
A2A	NAVY STAFF (ONR only)
A3	CHIEF OF NAVAL OPERATIONS
A5	NAVPERS & BUMED
A6	COMMANDANT MC (code LFF)
B2A	(JCS, NSA, DLA, DNA only)
B5	USCG only)
E3A	LAB ONR(Wash.DC only)
FA6	NAVAIRSTA LANT
FA7	NAVSTA LANT
FA10	SUB BASE LANT
FA18	PHIBASE LANT
FA23	NAVFAC LANT
FA32	CINCLANT CBU
FB6	NAVAIRFAC PAC
FB7	NAVAIRSTA PAC
FB10	NAVSTA PAC (Seattle only)
FB13	SUBASE PAC (Bangor only)
FB21	PHIBASE PAC
FB34	FLEACTPAC (Kadena,Sasebo only)
FB36	FAC PAC
FB48	SUPPFAC PAC
FC3	ACTIVITY EUR (London only)
FC5	SUPPORT ACTIVITY EUR
FC7	NAVAL STATION EUR
FC12	FLEET SUPPORT OFFICE
FC14	NAVAIRSTA EUR
FD1	OCEANOGRAPHY COMM
FE1	SECURITY GROUP HQ
FE2	SECURITY STATION
FE4	SECGRUACT (Edzell, Hanza Homestead, Sabana Seca, Sonoma, Winter Harbor only)
FF6	NAVAL OBSERVATORY
FF38	NAVAL ACADEMY
FG1	TELECOMMCOM
FG2	COMSTA (Balboa, Harold Holt, Nea Makri, Thurso, Stockton, Yokosuka, San Miguel only)
FG3	COMM UNIT (East Machias only)
FG6	CA MASTER STAS (Norfolk only)
FH1	COMNAVMEDCOM
FKA1A	SYSCOM, AIR
FKA1B	SYSCOM, ELEX
FKA1C	NAVFAC,(Code 04M2, 30 copies)
FKA1F	SYSCOM, SUP
FKA1G	SYSCOM, SEA
FKM8	SUPPLY ANNEX
FKM9	NSC (Oakland only)
FKM13	SHIPS PARTS & CONTROL CEN
FKM15	ASO (Philadelphia only)
FKN1	LANTDIV (100 copies) CHESDIV (50 copies) NORTHDIV (75 copies) PACDIV, SOUTHDIV, WESTDIV, (200 copies each)
FKN2	CBC

FKN3	OICC (6 copies each)
FKN5	PWC (5 copies each)
FKN7	ENERGY ENVIRON SUPP ACT
FKN10	SUPPORT ACT, NAV FAC
FKN11	CIVIL ENGINEERING LAB
FKP1B	WEAPONS STATION
FKP1E	UNDERSEA WARFARE ENGR STA
FKP1J	NAVAL ORDNANCE STATION
FKP1M	WEAPONS SUPPLY CENTER
FKP7	NAVAL SHIPYARDS
FKQ3A	ELECTRONIC STMS ENGR CEN
FKQ6A	AIR DEVELOPMENT CENTER
FKQ6B	COASTAL SYSTEMS CENTER
FKQ6C	OCEAN SYSTEMS CENTER
FKQ6E	SHIP RESEARCH & DEV.CEN.
FKQ6F	SURFACE WEAPONS CENTER
FKQ6G	UNDERWATER SYSTEMS CENTER
FKQ6H	NAVAL WEAPONS CENTER
FKR1A	AIR STATIONS R & D
FKR1B	AIR REWORK FACILITY
FKR3A	AIR ENGINEERING CENTER
FKR3H	AIR PROPULSION CENTER
FKR4B	MISSILE RANGE FACILITY
FKR5	AVIONICS CENTER
FKR7E	AVIATION LOGISTICS CENTER
FR3	AIR STATIONS
FR4	AIR FACILITIES
FR15	SUPPACT
FT1	CHIEF NAVAL ENGR. TRAINING
FT2	CHIEF NAVAL AIR TRAINING
FT6	AIR STATIONS CNET
FT13	AIR TECH TRAINING CEN
FT18	CONSTR. BATTALION UNIT CBC
FT19	ADMINCOM (San Diego only)
FT22	FLECOMBATRACEN (Va. Beach only)
FT28	EDUCATION & TRAINING CEN
FT31	TRAINING CENTER
FT37	SCHOOL CEC OFFICERS
FT55	SCHOOL SUPPLY CORPS
FT78	ED & TRNG PGM DEV CEN
V2	MARCORPS BARRACKS SUPPORT
V3	NC AIR BASE COMMAND
V5	MCAS (less Beaufort)
V5	MCAS (Beaufort only,5 cys)
V8	MC RECRUIT DEPOT
V12	MC DEVELOP.& EDUCATION CEN
V14	MC HEADQUARTERS BATTALION
V15	MC DISTRICT
V17	MC CAMP
V23	MC LOGISTICS BASE
V25	MC AIR GROUND COMBAT CEN

REMAINDER TO: RECEIVING OFFICER
Naval Publications and Forms Center
5801 Tabor Avenue, Phila., PA 19120
FOR IMMEDIATE STOCK

Information on how to obtain additional copies is contained in References.

RECORD OF DOCUMENT CHANGES

Instructions: DISCARD EXISTING SHEET MD INSERT THIS NEW RECORD OF DOCUMENT CHANGES.

This is an inventory of all changes made to this design manual. Each change is consecutively numbered, and each change page in the design manual includes the date of the change which issued it.

Change Number	Description of Change	Date of Change	Page Changed
1	Added new cover with revalidation date.	September 1986	Cover
	Added Record of Document Changes page.		-
	New Abstract.		iii
	Added to Foreword address for sending recommended changes and changed signature to RADM Jones.		v
	Added listing of DM-7 series.		vi
	Deleted Preface.		vii
	Deleted list of Design Manuals.		ix
	New Table of Contents.		vii-xiv
	New acknowledgments.		xv
	Deleted DM-9 and corrected title of DM 5.04 in Related Criteria.		7.1-1
	Changed date of Reference 13.		7.1-45
	Added NAVFAC DM's to Reference list.		7.1-47
	Updated Related Criteria listing.		7.1-49
	Added NAVFAC DM's and P-Pubs to Reference list.		7.1-116
	Updated Related Criteria listing.		7.1-117
	Changed AASHTO T174 to AASHTO T190.		7.1-126

RECORD OF DOCUMENT CHANGES (Continued)

Change Number	Description of Change	Date of Change	Page Changed
1	Changed DM-5 to DM-5.04 in paragraph 4.		7.1-154
	Added NAVFAC DM's to Reference list.		7.1-160
	Deleted NAVDOCKS P-81 and updated DM's in Related Criteria listing.		7.1-161
	Changed P/ to P/[pi].		7.1-175
	Added NAVFAC DM's to Reference list.		7.1-204
	Deleted DM-5.11 and updated DM's in Related Criteria list.		7.1-159
	Restated equations for Z+r, of q for the case of circle of wells penetrating artesian stratum.		7.1-284
	Added NAVFAC DM's and P-Pubs to Reference List.		7.1-307
	Changed Figure 15 to Figure 14.		7.1-346
	Added DD Form 1426.		
	Changed "plain" to "plane" in two places.		7.1-A-2

ABSTRACT

This manual covers the application of engineering principles by experienced engineers of soil mechanics in the design of foundations and earth structures for naval shore facilities. The contents include identification and classification of soil and rock, field exploration, testing, and instrumentation, laboratory testing, distribution of stresses including pressures on buried structures, analysis of settlement and volume expansion, seepage and drainage, and slope stability and protection.

PAGE iv INTENTIONALLY BLANK

FOREWORD

This design manual is one of a series developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command (NAVFACENGCOM), other Government agencies, and the private sector. This manual uses, to the maximum extent feasible, national professional society, association, and institute standards in accordance with NAVFACENGCOM policy. Deviations from these criteria should not be made without prior approval of NAVFACENGCOM Headquarters (Code 04).

Design cannot remain static any more than the rival functions it serves or the technologies it uses. Accordingly, recommendations for improvement are encouraged from within the Navy and from the private sector and should be furnished to Commander, Naval Facilities Engineering Command (Code 04B), 200 Stovall Street, Alexandria, VA 22332-2300.

This publication is certified as an official publication of the Naval Facilities Engineering Command and has been reviewed and approved in accordance with SECNAVINST 5600.16, Procedures Governing Review of the Department of the Navy (DN) Publications.

J. P. JONES, JR.
Rear Admiral, CEC, U. S. Navy
Commander
Naval Facilities Engineering Command

SOILS AND FOUNDATIONS DESIGN MANUALS

DM	Title
7.01	Soil Mechanics
7.02	Foundations and Earth Structures
7.03	Soil Dynamics, Peep Stabilization and Special Geotechnical Construction

CONTENTS

Page

FIGURES

Figure	Title	Page

CHAPTER 4

CHAPTER 5

Figure Title Page

CHAPTER 7

TABLES

ACKNOWLEDGMENTS

Figure or Table	Acknowledgments
Table 4, Chapter 1	Terzaghi, K., and Peck, R.B., Soil Mechanics in Engineneering Practice, John Wiley & Sons, New York, 1967.
Figure 3, Chapter 1	Broch, E., and Franklin, J.A., The Point Load Strength Test, International Journal of Rock Mechanics and Mineral Sciences, Pergamon Press, 1972.
Table 6, Chapter 1	Hough, B.K., Basic Soils Engineering, Ronald Press, New York, 1969.
Figure 10, Chapter 2	Acker Soil Sampling Catalog, Acker Drill Company, Scranton, PA.
Figure 14, Chapter 4	Mehta, M.R., and Veletsos, A.S., Stresses and Displacement in Layered Systems, Structural Research Series No. 178, University of Illinois, Urbana, IL.
Figures 18(a), 20, and 21, Chapter 4	Watkins, R.K., Buried Structures, Foundation Engineering Handbook, Winterkorn, H.F. and Fang, H.Y., ed., Chapter 27, Van Nostrand Reinhold Company, New York, 1975.
Figure 18(c), Chapter 4	Soft Ground Tunneling, (Company Brochure T-1), Commercial Shearing, Inc., Youngstown, OH., 1971.
Figure 18(e), Chapter 4	Concrete Pipe Design Manual, American Concrete Pipe Association, Vienna, VA., 1980.
Table 1, Chapter 4	Proctor, R.V., and White, T.L., Rock Tunneling with Steel Supports, Commercial Shearing, Inc., Youngstown, OH., 1977.
Figure 23 and Table 2, Chapter 4	Proctor, R.V., and White, T.L., Earth Tunneling with Steel Supports, Commercial Shearing, Inc., Youngstown, OH., 1977.
Figure 1, Chapter 7	Lambe, T.W., and Whitman, R.V., Soil Mechanics, John Wiley & Sons, Inc., New York, 1969.
Figures 2 and 3, Chapter 7	Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics Series No. 46, Harvard University, Cambridge, MA.

CHAPTER 1. IDENTIFICATION ID CLASSIFICATION OF SOIL AND ROCK

Section 1. INTRODUCTION

1. SCOPE. This chapter presents criteria for soil and rock identification and classification plus information on their physical engineering properties. Common soils and rock are discussed as well as special materials such as submarine soils and coral, saprolitic soils, lateritic soils, expansive and collapsing soils, cavernous limestone, quick clay, permafrost and hydraulically placed fills.

2. RELATED CRITERIA. For additional criteria on the classification and identification of soil and rock, see the following sources:

Subject	Source
Pavements...	NAVFAC DM-5.04
Airfield Pavement.......................................	NAVFAC DM-21 Series

Section 2. SOIL DEPOSITS

1. GEOLOGIC ORIGIN AND MODE OF OCCURRENCE.

a. Principal Soil Deposits. See Table 1 for principal soil deposits grouped in terms of origin (e.g., residual, colluvial, etc.) and mode of occurrence (e.g., fluvial, lacustrine, etc.).

b. Importance. A geologic description assists in correlating experiences between several sites, and in a general sense, indicates the pattern of strata to be expected prior to making a field investigation (test borings, etc.). Soils with similar origin and mode of occurrence are expected to have comparable if not similar engineering properties. For quantitative foundation analysis, a geological description is inadequate and more specific classification is required. For sources of information on the physical geology of the United States, see Chapter 2. A study of references on local geology should precede a major subsurface exploration program.

c. Soil Horizon. Soil horizons are present in all sedimentary soils and transported soils subject to weathering. The A horizon contains the maximum amount of organic matter; the underlying B horizon contains clays, sesquioxides, and small amounts of organic matter. The C horizon is partly weathered parent soil or rock and the D horizon is unaltered parent soil and rock.

TABLE 1
Principal Soil Deposits

Major Division	Principal Soil Deposits	Pertinent Engineering Characteristics
SEDIMENTARY SOILS -		
Residual		
Material formed by disintegration of underlying parent rock or partially indurated material.	Residual sands and fragments of gravel size formed by solution and leaching of cementing material, leaving the more resistant particles; commonly quartz	Generally favorable foundation conditions
	Residual clays formed by decomposition of silicate rocks, disintegration of shales, and solution of carbonates in limestone. With few exceptions becomes more compact, rockier, and less weathered with increasing depth. At intermediate stage may reflect composition, structure, and stratification of parent rock.	Variable properties requiring detailed investigation. Deposits present favor able foundation conditions except in humid and tropical climates, where depth and rate of weathering are very great.
Organic		
Accumulation of highly organic material formed in place by the growth and subsequent decay of plant life.	Peat. A somewhat fibrous aggregate of decayed and decaying vegetation matter having a dark color and odor of decay.	Very compressible. Entirely unsuitable for supporting building foundations,
	Muck. Peat deposits which have advanced In stage of decomposition to such extent that the botanical character is no longer evident.	

TABLE 1 (continued)
Principal Soil Deposits

Major Division	Principal Soil Deposits	Pertinent Engineering Characteristics
TRANSPORTED SOILS -		
Alluvial		
Material transported and deposited by running water.	<u>Floodplain deposits</u>. Deposits laid down by a stream within that portion of its valley subject to inundation by floodwaters.	
	<u>Point bar</u>. Alternating deposits of arcuate ridges and swales (lows) formed on the inside or convex bank of mitigating river bends. Ridge deposits consist primarily of silt and sand, swales are clay-filled.	Generally favorable foundation conditions; however, detailed investigations are necessary to locate discontinuities. Flow slides may be a problem along riverbanks. Soils are quite pervious.
	<u>Channel fill</u>. Deposits laid down in abandoned meander loops isolated when rivers shorten their courses. Composed primarily of clay; however, silty and sandy soils are found at the upstream and downstream ends.	Fine-grained soils are usually compressible. Portions may be very heterogeneous. Silty soils generally present favorable foundation conditions.
	<u>Backswamp</u>. The prolonged accumulation of floodwater sediments in flood basins bordering a river. Materials are generally clays but tend to become more silty near riverbank.	Relatively uniform in a horizontal direction. Clays are usually subjected to seasonal volume changes.
	<u>Alluvial Terrace deposits</u>. Relatively narrow, flat-surfaced, river-flanking remnants of floodplain deposits formed by entrenchment of rivers and associated processes.	Usually drained, oxidized. Generally favorable foundation conditions.

TABLE 1 (continued)
Principal Soil Deposits

Major Division	Principal Soil Deposits	Pertinent Engineering Characteristics
(cont'd) Materials transported and deposited by running water.	Estuarine deposits. Mixed deposits of marine and alluvial origin laid down in widened channels at mouths of rivers and influenced by tide of body of water into which they are deposited.	Generally fine-grained and compressible. Many local variations in soil conditions.
	Alluvial-Lacustrine deposits. Material deposited within lakes (other than those associated with glaciation) by waves, currents, and organo-chemical processes. Deposits consist of unstratified organic clay or clay in central portions of the lake and typically grade to stratified silts and sands in peripheral zones.	Usually very uniform in horizontal direction. Fine-grained soils generally compressible.
	Deltaic deposits. Deposits formed at the mouths of rivers which result in extension of the shoreline.	Generally fine-grained and compressible. Many local variations in soil condition.
	Piedmont deposits. Alluvial deposits at foot of hills or mountains. Extensive plains or alluvial fans.	Generally favorable foundation conditions.
Aeolian		
Material transported and deposited by wind.	Loess. A calcareous, unstratified deposit of silts or sandy or clayey silt traversed by a network of tubes formed by root fibers now decayed.	Relatively uniform deposits characterized by ability to stand in vertical cuts. Collapsible structure. Deep weathering or saturation can modify characteristics.
	Dune sands. Mounds, ridges, and hills of uniform fine sand characteristically exhibiting rounded grains.	Very uniform grain size; may exist in relatively loose condition.

TABLE 1 (continued)
Principal Soil Deposits

Major Division	Principal Soil Deposits	Pertinent Engineering Characteristics
Glacial		
Material transported and deposited by glaciers, or by meltwater from the glacier.	<u>Glacial till</u>. An accumulation of debris, deposited beneath, at the side (lateral moraines), or at the lower limit of a glacier (terminal moraine). Material lowered to ground surface in an irregular sheet by a melting glacier is known as a ground moraine.	Consists of material of all sizes in various proportions from boulder and gravel to clay. Deposits are unstratified. Generally present favorable foundation conditions; but, rapid changes in conditions are common.
	<u>Glacio-Fluvial deposits</u>. Coarse and fine-grained material deposited by streams of meltwater from glaciers. Material deposited on ground surface beyond terminal of glacier is known as an outwash plain. Gravel ridges known as kames and eskers.	Many local variations. Generally present favorable foundation conditions.
	<u>Glacio-Lacustrine deposits</u>. Material deposited within lakes by meltwater from glaciers. Consisting of clay in central portions of lake and alternate layers of silty clay or silt and clay (varved clay) in peripheral zones.	Very uniform in a horizontal direction.
Marine		
Material transported and deposited by ocean waves and currents in shore and offshore areas.	<u>Shore deposits</u>. Deposits of sands and/or gravels formed by the transporting, destructive, and sorting action of waves on the shoreline.	Relatively uniform and of moderate to high density.
	<u>Marine clays</u>. Organic and inorganic deposits of fine-grained material.	Generally very uniform in composition. Compressible and usually very sensitive to remolding.

TABLE 1 (continued)
Principal Soil Deposits

Major Division	Principal Soil Deposits	Pertinent Engineering Characteristics
Colluvial		
Material transported and deposited by gravity.	<u>Talus</u>. Deposits created by gradual accumulation of unsorted rock fragments and debris at base of cliffs.	Previous movement indicates possible future difficulties. Generally unstable foundation conditions.
	<u>Hillwash</u>. Fine colluvium consisting of clayey sand, sand silt, or clay.	
	<u>Landslide deposits</u>. Considerable masses of soil or rock that have slipped down, more or less as units, from their former position on steep slopes.	
Pyroclastic		
Material ejected from volcanoes and transported by gravity, wind and air.	<u>Ejecta</u>. Loose deposits of volcanic ash, lapilli, bombs, etc.	Typically shardlike particles of silt size with larger volcanic debris. Weathering and redeposition produce highly plastic, compressible clay. Unusual and difficult foundation conditions.
	<u>Pumice</u>. Frequently associated with lava flows and mud flows, or may be mixed with nonvolcanic sediments.	

Section 3. SOIL IDENTIFICATION

1. REQUIREMENTS. A complete engineering soil identification includes: (a) a classification of constituents, (b) the description of appearance and structural characteristics, and (c) the determination of compactness or consistency in situ.

a. Field Identification. Identify constituent materials visually according to their grain size, and/or type of plasticity characteristics per ASTM Standard D2488, Description of Soils (Visual-Manual Procedure).

(1) Coarse-Grained Soils. Coarse-grained soils are those soils where more than half of particles finer than 3-inch size can be distinguished by the naked eye. The smallest particle that is large enough to be visible corresponds approximately to the size of the opening of No. 200 sieve used for laboratory identification. Complete identification includes grain size, color, and/or estimate of compactness.

(a) Color. Use color that best describes the sample. If there are two colors describe both colors. If there are more than two distinct colors, use multi-colored notation.

(b) Grain Size. Identify components and fractions in accordance with Table 2 - Coarse-Grained Soils.

(c) Grading. Identify both well graded or poorly graded sizes as explained in Table 3, under Supplementary Criteria for Visual Identification.

(d) Assigned Group Symbol. Use Table 3 for estimate of group symbols based on the Unified Classification System.

(e) Compactness. Estimate compactness in situ by measuring resistance to penetration of a selected penetrometer or sampling device (see Chapter 2). If the standard penetration test is performed, determine the number of blows of a 140 pound hammer falling 30 inches required to drive a 2-inch OD, 1-3/8 inch ID split barrel sampler 1 foot. The number of blows thus obtained is known as the standard penetration resistance, N. The split barrel is usually driven 18 inches. The penetration resistance is based on the last 12 inches.

1) Description Terms. See Figure 1 (Reference 1, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), by the Departments of the Army and Air Force) for descriptive terms of compactness of sand. Figure 1 is applicable for normally consolidated sand.

2) Compactness Based on Static Cone Penetration Resistance, q+c,. Reference 2, Cone Resistance as Measure of Sand Strength, by Mitchell and Lunne, provides guidance for estimating relative density with respect to the cone resistance. If q+c, and N values are measured during the field exploration, a q+c,-N correlation could be made, and Figure 1 is used to describe compactness. If N is not measured, but q+c, is measured, then use

TABLE 2
Visual Identification of Samples

Definitions of Soil Components and Fractions

1. Grain Size

Material	Fraction	Sieve Size
Boulders		12"+
Cobbles		3" - 12"
Gravel	coarse	3/4" - 3"
	fine	No. 4 to 3/4"
Sand	coarse	No. 10 to No. 4
	medium	No. 40 to No. 10
	fine	No. 200 to No. 40
Fines (Silt & Clay)		Passing No. 200

2. Coarse- and Fine-Grained Soils

Descriptive Adjective	Percentage Requirement
trace	1 - 10%
little	10 - 20%
some	20 - 35%
and	35 - 50%

3. Fine-Grained Soils. Identify in accordance with plasticity characteristics, dry strength, and toughness as described in Table 3.

	Descriptive Term	Thickness
Stratified Soils	alternating	
	thick	
	thin	
	with	
	parting	- 0 to 1/16" thickness
	seam	- 1/16 to 1/2" thickness
	layer	- 1/2 to 12" thickness
	stratum	- greater than 12" thickness
	varved Clay	- alternating seams or layers of sand, silt and clay
	pocket	- small, erratic deposit, usually less than 1 foot
	lens	- lenticular deposit
	occasional	- one or less per foot of thickness
	frequent	- more than one per foot of thickness

TABLE 3
Unified Soil Classification System

Primary Divisions for Field and Laboratory Identification			Group Symbol	Typical Names	Laboratory Classification Criteria	Supplementary Criteria For Visual Identification
Coarse-grained soils. (More than half of material finer than 3-inch sieve is larger than No. 200 sieve size.)	Gravel. (More than half of the coarse fraction is larger than No. 4 sieve size about 1/4 inch.)	Clean gravels. (Less than 5% of material smaller than No. 200 sieve size.)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.*	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4. $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3.	Wide range in grain size and substantial amounts of all intermediate particle size.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.*	Not meeting both criteria for GW.	Predominantly one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded).

* Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.

TABLE 3 (continued)
Unified Soil Classification System

Primary Divisions for Field and Laboratory Identification			Group Symbol	Typical Names	Laboratory Classification Criteria		Supplementary Criteria For Visual Identification
.....do....	.....do....	Gravels with fines. (More than 12% of material smaller than No. 200 sieve size.)*	GM	Silty gravels, and gravel-sand-silt mixtures.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 & 7 is borderline case GM-GC	Nonplastic fines or fines of low plasticity.
			GC	Clayey gravels, and gravel-sand-clay mixtures.	Atterberg limits above "A" line, and PI greater than 7.		Plastic fines.
....do.....	Sands. (More than half of the coarse fraction is smaller than No. 4 sieve size.)	Clean sands. (Less than 5% of material smaller than No. 200 sieve size.)	SW	Well graded sands, gravelly sands, little or no fines.*	$C_u = \frac{D_{60}}{D_{10}}$ greater than 6. $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3.		Wide range in grain sizes and substantial amounts of all intermediate particle sizes.
			SP	Poorly graded sands and gravelly sands, little or no fines.*	Not meeting both criteria for SW.		Predominately one size (uniformly graded) or a range of sizes with some intermediate sizes missing (gap graded).

* Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.

TABLE 3 (continued)
Unified Soil Classification System

Primary Divisions for Field and Laboratory Identification			Group Symbol	Typical Names	Laboratory Classification Criteria		Supplementary Criteria For Visual Identification
.....do....	.....do....	Sands with fines. (More than 12% of material smaller than No. 200 sieve size.)*	SM	Silty sands, sand-silt mixtures.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 and 7 is borderline case SM-SC.	Nonplastic fines or fines of low plasticity.
			SC	Clayey sands, sand-clay mixtures.	Atterberg limits above "A" line with PI greater than 7.		Plastic fines.

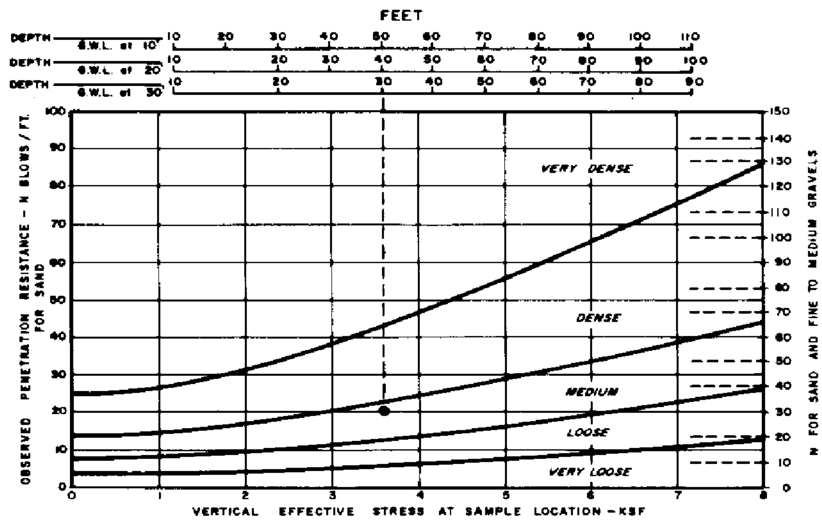
* Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SC, etc.

TABLE 3 (continued)
Unified Soil Classification System

Primary Divisions for Field and Laboratory Identification		Group Symbol	Typical Names	Laboratory Classification Criteria		Supplementary Criteria For Visual Identification		
						Dry Strength	Reaction to Shaking	Toughness Near Plastic Limit
Fine-grained soils. (More than half of material is smaller than No. 200 sieve size.) (Visual: more than half of particles are so fine that they cannot be seen by naked eye.)	Silts and clays. (Liquid limit less than 50.)	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands.	Atterberg limits below "A" line, or PI less than 4.	Atterberg limits above "A" line with PI between 4 and 7 is borderline case ML-CL.	None to slight	Quick to slow	None
	........do........	CL	Inorganic clays of low to medium plasticity; gravelly clays, silty clays, sandy clays, lean clays.	Atterberg limits above "A" line, with PI greater than 7.		Medium to high	None to very slow	Medium
	........do........	OL	Organic silts and organic silt-clays of low plasticity.	Atterberg limits below "A" line.		Slight to medium	Slow	Slight

TABLE 3 (continued)
Unified Soil Classification System

Primary Divisions for Field and Laboratory Identification		Group Symbol	Typical Names	Laboratory Classification Criteria	Supplementary Criteria For Visual Identification		
					Dry Strength	Reaction to Shaking	Toughness Near Plastic Limit
.....do....	Silts and clays. (Liquid limit greater than 50.)	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts.	Atterberg limits below "A" line.	Slight to medium	Slow to none	Slight to medium
	........do........	CH	Inorganic clays of high plasticity, fat clays.	Atterberg limits above "A" line.	High to very high	None	High
	........do........	OH	Organic clays of medium to high plasticity.	Atterberg limit below "A" line	Medium to high	None to very slow	Slight to medium
.....do....	Highly organic soils............	Pt	Peat, muck and other highly organic soils.	High ignition loss, LL and PI decrease after drying.	Organic color and odor, spongy feel, frequently fibrous texture.		



Example:

Blow count in sand at a depth of 40 ft = 20
Depth of Groundwater Table = 20 ft
Compactness ~ medium

FIGURE 1
Estimated Compactness of Sand from Standard Penetration Test

N = q+c,/4 for sand and fine to medium gravel and N = q+c,/5 for sand, and use Figure 1 for describing compactness.

(f) Describe, if possible, appearance and structure such as angularity, cementation, coatings, and hardness of particles.

(g) Examples of Sample Description:

Medium dense, gray coarse to fine SAND, trace silt, trace fine gravel (SW). Dry, dense, light brown coarse to fine SAND, some silt (SM).

(2) Fine-Grained Soils. Soils are identified as fine-grained when more than half of the particles are finer than No. 200 sieve (as a field guide, such particles cannot be seen by the naked eye). Fine-grained soils cannot be visually divided between silt and clay, but are distinguishable by plasticity characteristics and other field tests.

(a) Field Identification. Identify by estimating characteristics in Table 3.

(b) Color. Use color that best describes the sample. If two colors are used, describe both colors. If there are more than two distinct colors, use multi-colored notation.

(c) Stratification. Use notations in Table 2.

(d) Appearance and Structure. These are best evaluated at the time of sampling. Frequently, however, it is not possible to give a detailed description of undisturbed samples in the field. Secondary structure in particular may not be recognized until an undisturbed sample has been examined and tested in the laboratory. On visual inspection, note the following items:

1) Ordinary appearance, such as color; moisture conditions, whether dry, moist, or saturated; and visible presence of organic material.

2) Arrangement of constituent materials, whether stratified, varved, or heterogeneous; and typical dip and thickness of lenses or varves.

3) Secondary structure, such as fractures, fissures, slickensides, large voids, cementation, or precipitates in fissures or openings.

(e) General Field Behavior.

1) Clays. Clays exhibit a high degree of dry strength in a small cube allowed to dry, high toughness in a thread rolled out at plastic limit, and exude little or no water from a small pat shaken in the hand.

2) Silts. Silts have a low degree of dry strength and toughness, and dilate rapidly on shaking so that water appears on the sample surface.

3) Organic Soils. Organic soils are characterized by dark colors, odor of decomposition, spongy or fibrous texture, and visible particles of vegetal matter.

(f) Consistency. Describe consistency in accordance with Table 4 (Reference 3, Soil Mechanics in Engineering Practice, by Terzaghi and Peck). Use a pocket penetrometer or other shear device to check the consistency in the field.

(g) Assignment of Group Symbol. Assign group symbol in accordance with Table 3.

(h) Examples of Sample Description:

Very stiff brown silty CLAY (CL), wet
Stiff brown clayey SILT (ML), moist
Soft dark brown organic CLAY (OH), wet.

Section 4. SOIL CLASSIFICATION AND PROPERTIES

1. REFERENCE. Soil designations in this manual conform to the Unified Soil Classification (see Table 3) per ASTM D2487, Classification of Soil for Engineering Purposes.

2. UTILIZATION. Classify soils in accordance with the Unified System and include appropriate group symbol in soil descriptions. (See Table 3 for elements of the Unified System.) A soil is placed in one of 15 categories or as a borderline material combining two of these categories. Laboratory tests may be required for positive identification. Use the system in Table 2 for field soil description and terminology.

a. Sands and Gravels. Sands are divided from gravels on the No. 4 sieve size, and gravels from cobbles on the 3-inch size. The division between fine and medium sands is at the No. 40 sieve, and between medium and coarse sand at the No. 10 sieve.

b. Silts and Clays. Fine-grained soils are classified according to plasticity characteristics determined in Atterberg limit tests. Categories are illustrated on the plasticity chart in Figure 2.

c. Organic Soils. Materials containing vegetable matter are characterized by relatively low specific gravity, high water content, high ignition loss, and high gas content. Decrease in liquid limit after oven-drying to a value less than three-quarters of the original liquid limit is a definite indication of an organic soil. The Unified Soil Classification categorizes organic soils based on the plotted position on the A-line chart as shown in Figure 2. However, this does not describe organic soils completely.

TABLE 4
Guide for Consistency of Fine-Grained Soils

SPT Penetration (blows/foot)	Estimated Consistency	Estimated Range of Unconfined Compressive Strength tons/sq. ft.
<2	Very soft (extruded between fingers when squeezed)	<0.25
2 - 4	Soft (molded by light finger pressure)	0.25 - 0.50
4 - 8	Medium (molded by strong finger pressure)	0.50 - 1.00
8 - 15	Stiff (readily indented by thumb but penetrated with great effort)	1.00 - 2.00
15 - 30	Very stiff (readily indented by thumbnail)	2.00 - 4.00
>30	Hard (indented with difficulty by thumbnail)	>4.00

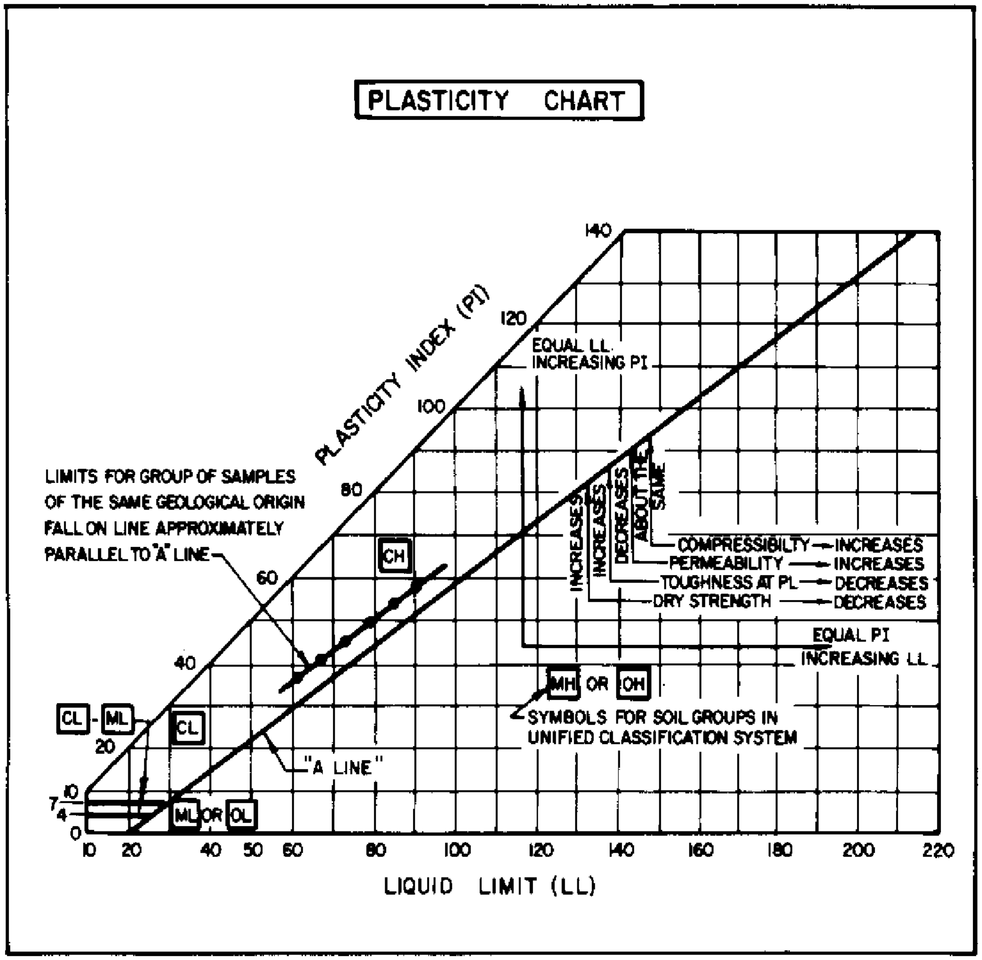


FIGURE 2
Utilization of Atterberg Plasticity Limits

Therefore, Table 5 (Reference 4, unpublished work by Ayers and Plum) is provided for a more useful classification of organic soils.

For the characteristics of the Unified Soil Classification System pertinent to roads and airfields, see NAVFAC DM-5.4.

3. TYPICAL PROPERTIES. Some typical properties of soils classified by the Unified System are provided in Table 6 (Reference 5, Basic Soils Engineering, by Hough). More accurate estimates should be based on laboratory and/or field testing, and engineering evaluation.

Section 5. ROCK CLASSIFICATION AND PROPERTIES

1. VISUAL CLASSIFICATION. Describe the rock sample in the following sequence:

a. Weathering Classification. Describe as fresh, slightly weathered, etc. in accordance with Table 7 (Reference 6, Suggested Methods of the Description of Rock Masses, Joints and Discontinuities, by ISRM Working Party).

b. Discontinuity Classification. Describe spacing of discontinuities as close, wide, etc., in accordance with Table 8. In describing structural features, describe rock mass as thickly bedded or thinly bedded, in accordance with Table 8. Depending on project requirements, identify the form of joint (stepped, smooth, undulating, planar, etc.), its dip (in degrees), its surface (rough, smooth, slickensided), its opening (giving width), and its filling (none, sand, clay, breccia, etc.).

c. Color and Grain Size. Describe with respect to basic colors on rock color chart (Reference 7, Rock Color Chart, by Geological Society of America). Use the following term to describe grain size:

(1) For Igneous and Metamorphic Rocks:

coarse-grained - grain diameter >5mm

medium-grained - grain diameter 1 - 5mm

fine-grained - grain diameter <1mm

aphanitic - grain size is too small to be perceived by unaided eye

glassy - no grain form can be distinguished.

(2) For Sedimentary Rocks

coarse-grained - grain diameter >2mm

medium-grained - grain diameter = 0.06 - 2mm

TABLE 5
Soil Classification for Organic Soils

Category	Name	Organic Content (% by wt.)	Group Symbols (See Table 3)	Distinguishing Characteristics For Visual Identification	Range of Laboratory Test Values
ORGANIC MATTER	FIBROUS PEAT (woody, mats, etc.)	75 to 100% Organics either visible or inferred	Pt	Light weight, spongy and often elastic at w_n--shrinks considerably on air drying. Much water squeezes from sample.	w_n--500 to 1200% γ--60 to 70 pcf G--1.2 to 1.8 $C_c/(1+e_o)$=.4+
	FINE GRAINED PEAT (amor-phous)			Light weight, spongy but not often elastic at w_n--shrinks considerably on air drying. Much water squeezes from sample.	w_n--400 to 800% LL--400 to 900% PI--200 to 500 γ--60 to 70 pcf G--1.2 to 1.8 $C_c/(1+e_o)$=.35 to .4+
HIGHLY ORGANIC SOILS	Silty Peat	30 to 75% Organics either visible or inferred	Pt	Relatively light weight, spongy. Thread usually weak and spongy near PL Shrinks on air drying; medium dry strength. Usually can squeeze water from sample readily--slow dilatency.	w_n--250 to 500% LL--250 to 600% PI--150 to 350 γ--65 to 90 pcf G--1.8 to 2.3 $C_c/(1+e_o)$=.3 to .4
	Sandy Peat			Sand fraction visible. Thread weak and friable near PL; shrinks on air drying; low dry strength. Usually can squeeze water from sample readily--high dilatency--"gritty."	w_n--100 to 400% LL--150 to 300% (plot below A line) PI--50 to 150 γ--70 to 100 pcf G--1.8 to 2.4 $C_c/(1+e_o)$=.2 to .3

TABLE 5 (continued)
Soil Classification for Organic Soils

Category	Name	Organic Content (% by wt.)	Group Symbols (See Table 3)	Distinguishing Characteristics For Visual Identification	Range of Laboratory Test Values
ORGANIC SOILS	Clayey ORGANIC SILT	5 to 30% Organics either visible or inferred	OH	Often has strong H_2S odor. Thread may be tough depending on clay fraction. Medium dry strength, slow dilatency.	w_n--65 to 200% LL--65 to 150% (usually plot at or near A line) PI-- 50 to 150 γ --70 to 100 pcf G--2.3 to 2.6 $C_c/(1+e_o)$=.20 to .35
	Organic SAND or SILT		OL	Threads weak and friable near PL--or may not roll at all. Low dry strength; medium to high dilatency.	w_n--30 to 125% LL--30 to 100% (usually plot well below A line) PI--non-plastic to 40 γ --90 to 110 pcf G--2.4 to 2.6 $C_c/(1+e_o)$=.1 to .25
SLIGHTLY ORGANIC SOILS	SOIL FRACTION add slightly Organic	Less than 5% Organics combined visible and inferred	Depend upon inorganic fraction	Depend upon the characteristics of the inorganic fraction.	Depend upon inorganic fractions.

TABLE 6
Typical Values of Soil Index Properties

	Particle Size and Gradation				Voids(1)					Unit Weight(2) (lb./cu.ft.)						
	Approximate Size Range (mm)		Approx. D_{10} (mm)	Approx. Range Uniform Coefficient C_u	Void Ratio			Porosity (%)		Dry Weight			Wet Weight		Submerged Weight	
	D_{max}	D_{min}			e_{max} loose	e_{cr}	e_{min} dense	n_{max} loose	n_{min} dense	Min loose	100% Mod. AASHO	Max dense	Min loose	Max dense	Min loose	Max dense
GRANULAR MATERIALS																
Uniform Materials																
a. Equal spheres (theoretical values)	-	-	-	1.0	0.92	-	0.35	47.6	26	-	-	-	-	-	-	-
b. Standard Ottawa SAND	0.84	0.59	0.67	1.1	0.80	0.75	0.50	44	33	92	-	110	93	131	57	69
c. Clean, uniform SAND (fine or medium)	-	-	-	1.2 to 2.0	1.0	0.80	0.40	50	29	83	115	118	84	136	52	73
d. Uniform, inorganic SILT	0.05	0.005	0.012	1.2 to 2.0	1.1	-	0.40	52	29	80	-	118	81	136	51	73
Well-graded Materials																
a. Silty SAND	2.0	0.005	0.02	5 to 10	0.90	-	0.30	47	23	87	122	127	88	142	54	79
b. Clean, fine to coarse SAND	2.0	0.05	0.09	4 to 6	0.95	0.70	0.20	49	17	85	132	138	86	148	53	86
c. Micaceous SAND	-	-	-	-	1.2	-	0.40	55	29	76	-	120	77	138	48	76
d. Silty SAND & GRAVEL	100	0.005	0.02	15 to 300	0.85	-	0.14	46	12	89	-	146(3)	90	155(3)	56	92
MIXED SOILS																
Sandy or Silty CLAY	2.0	0.001	0.003	10 to 30	1.8	-	0.25	64	20	60	130	135	100	147	38	85
Skip-graded Silty CLAY with stones or rk fgmts	250	0.001	-	-	1.0	-	0.20	50	17	84	-	140	115	151	53	89
Well-graded GRAVEL, SAND, SILT & CLAY mixture	250	0.001	0.002	25 to 1000	0.70	-	0.13	41	11	100	140	148(4)	125	156(4)	62	94
CLAY SOILS																
CLAY (30%-50% clay sizes)	0.05	0.5μ	0.001	-	2.4	-	0.50	71	33	50	105	112	94	133	31	71
Colloidal CLAY (-0.002 mm: 50%)	0.01	10Å	-	-	12	-	0.60	92	37	13	90	106	71	128	8	66
ORGANIC SOILS																
Organic SILT	-	-	-	-	3.0	-	0.55	75	35	40	-	110	87	131	25	69
Organic CLAY (30% - 50% clay sizes)	-	-	-	-	4.4	-	0.70	81	41	30		100	81	125	18	62

TABLE 6 (continued)
Typical Values of Soil Index

(1). Granular materials may reach e_{max} when dry or only slightly moist. Clays can reach e_{max} only when fully saturated.

(2) Granular materials reach minimum unit weight when at e_{max} and with hygroscopic moisture only. The unit submerged weight of any saturated soil is the unit weight minus the unit weight of water.

(3) Applicable for very compact glacial till. Unusually high unit weight values for tills are sometimes due to not only an extremely compact condition but to unusually high specific gravity values.

(4) Applicable for hardpan.

General Note: Tabulation is based on G = 2.65 for granular soil, G = 2.7 for clays, and G = 2.6 for organic soils.

TABLE 7
Weathering Classification

GRADE	SYMBOL	DIAGNOSTIC FEATURES
Fresh	F	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	WS	Slight discoloration inwards from open fractures, otherwise similar to F.
Moderately Weathered	WM	Discoloration throughout. Weaker minerals such as feldspar decomposed. Strength somewhat less than fresh rock but cores cannot be broken by hand or scraped by knife. Texture preserved.
Highly Weathered	WH	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Completely Weathered	WC	Minerals decomposed to soil but fabric and structure preserved (Saprolite). Specimens easily crumbled or penetrated.
Residual Soil	RS	Advanced state of decomposition resulting in plastic soils. Rock fabric and structure completely destroyed. Large volume change.

TABLE 8
Discontinuity Spacing

Description for Structural Features: Bedding, Foliation, or Flow Banding	Spacing	Description for Joints, Faults or Other Fractures
Very thickly (bedded, foliated, or banded)	More than 6 feet	Very widely (fractured or jointed)
Thickly	2 - 6 feet	Widely
Medium	8 - 24 inches	Medium
Thinly	2-1/2 - 8 inches	Closely
Very thinly	3/4 - 2-1/2 inches	Very closely

Description for Micro-Structural Features: Lamination, Foliation, or Cleavage	Spacing	Description for Joints, Faults or Other Fractures
Intensely (laminated, foliated, or cleaved)	1/4 - 3/4 inch	Extremely close
Very intensely	Less than 1/4 inch	

fine-grained - grain diameter = 0.002 - 0.06mm

very fine-grained - grain diameter <0.002mm

(3) Use lOX hand lens if necessary to examine rock sample.

d. Hardness Classification. Describe as very soft, soft, etc. in accordance with Table 9 (from Reference 5), which shows range of strength values of intact rock associated with hardness classes.

e. Geological Classification. Identify the rock by geologic name and local name (if any). A simplified classification is given in Table 10. Identify subordinate constituents in rock sample such as seams or bands of other type of minerals, e.g., dolomitic limestone, calcareous sandstone, sandy limestone, mica schist. Example of typical description:

Fresh gray coarse moderately close fractured Mica Schist.

2. CLASSIFICATION BY FIELD MEASUREMENTS AND STRENGTH TESTS.

a. Classification by Rock Quality Designation and Velocity Index.

(1) The Rock Quality Designation (RQD) is only for NX size core samples and is computed by summing the lengths of all pieces of core equal to or longer than 4 inches and dividing by the total length of the coring run. The resultant is multiplied by 100 to get RQD in percent. It is necessary to distinguish between natural fractures and those caused by the drilling or recovery operations. The fresh, irregular breaks should be ignored and the pieces counted as intact lengths. Depending on the engineering requirements of the project, breaks induced along highly anisotropic planes, such as foliation or bedding, may be counted as natural fractures. A qualitative relationship between RQD, velocity index and rock mass quality is presented in Table 11 (Reference 8, Predicting Insitu Modulus of Deformation Using Rock Quality Indexes, by Coon and Merritt).

(2) The velocity index is defined as the square of the ratio of the field compressional wave velocity to the laboratory compressional wave velocity. The velocity index is typically used to determine rock quality using geophysical surveys. For further guidance see Reference 9, Design of Surface and Near Surface Construction in Rock, by Deere, et al.

b. Classification by Strength.

(1) Uniaxial Compressive Strength and Modulus Ratio. Determine the uniaxial compressive strength in accordance with ASTM Standard D2938, Unconfined Compressive Strength of Intact Rock Core Specimens. Describe the strength of intact sample tested as weak, strong, etc., in accordance with Figure 3 (Reference 10, The Point Load Strength Test, by Broch and Franklin).

(2) Point Load Strength. Describe the point load strength of specimen tested as low, medium, etc. in accordance with Figure 3. Point load strength tests are sometimes performed in the field for larger projects where rippability and rock strength are critical design factors. This simple field test can be performed on core samples and irregular rock specimens. The point

TABLE 9
Hardness Classification of Intact Rock

CLASS	HARDNESS	FIELD TEST	APPROXIMATE RANGE OF UNIAXIAL COMPRESSION STRENGTH kg/cm.2- (tons/ft.2-)
I	Extremely hard	Many blows with geologic hammer required to break intact specimen.	>2000
II	Very hard	Hand held specimen breaks with hammer end of pick under more than one blow.	2000- 1000
III	Hard	Cannot be scraped or peeled with knife, hand held specimen can be broken with single moderate blow with pick.	1000 - 500
IV	Soft	Can just be scraped or peeled with knife. Indentations 1mm to 3mm show in specimen with moderate blow with pick.	500 - 250
V	Very soft	Material crumbles under moderate blow with sharp end of pick and can be peeled with a knife, but is too hard to hand-trim for triaxial test specimen.	250 - 10

TABLE 10
Simplified Rock Classification

COMMON IGNEOUS ROCKS

Color	Light		Intermediate	Dark	
Principal Mineral / Texture	Quartz & Feldspar Other Minerals Minor	Feldspar	Feldspar & Hornblende	Augite and Feldspar	Augite Hornblende, Olivine
Coarse, Irregular, Crystalline	Pegmatite	Syenite pegmatite	Diorite pegmatite	Gabbro pegmatite	
Coarse and Medium Crystalline	Granite	Syenite	Diorite	Gabbro	Peridotite
			Dolerite		
Fine Crystalline	Aplite			Diabase	
Aphanitic	Felsite			Basalt	
Glassy	Volcanic glass			Obsidian	
Porous (Gas Openings	Pumice		Scoria or vesicular basalt		
Fragmental	Tuff (fine), breccia (coarse), cinders (variable)				

TABLE 10 (continued)
Simplified Rock Classification

COMMON SEDIMENTARY ROCKS

Group	Grain Size	Composition		Name
Clastic	Mostly Coarse Grains	Rounded pebbles in medium-grained matrix		Conglomerate
		Angular coarse rock fragments, often quite variable		Breccia
	More than 50% of medium grains	Medium quartz grains	Less than 10% of other minerals	Siliceous sandstone
			Appreciable quantity of clay minerals	Argillaceous sands tone
			Appreciable quantity of calcite	Calcareous sandstone
			Over 25% feldspar	Arkose
			25-50% feldspar and darker minerals	Graywacke
	More than 50% fine grain size	Fine to very fine quartz grains with clay minerals		Siltstone (if laminated, shale)
		Microscopic clay minerals	<10% other minerals	Shale
			Appreciable calcite	Calcareous shale
			Appreciable carbon carbonaceous material	Carbonaceous shale
			Appreciable iron oxide cement	Ferruginous shale

TABLE 10 (continued)
Simplified Rock Classification

COMMON SEDIMENTARY ROCKS

Group	Grain Size	Composition	Name
Organic	Variable	Calcite and fossils	Fossiliferous limestone
	Medium to microscopic	Calcite and appreciable dolomite	Dolomite limestone or dolomite
	Variable	Carbonaceous material	Bituminous coal
Chemical	Microscopic	Calcite	Limestone
		Dolomite	Dolomite
		Quartz	Chert, Flint, etc.
		Iron compounds with quartz	Iron formation
		Halite	Rock salt
		Gypsum	Rock gypsum

TABLE 10 (continued)
Simplified Rock Classification

COMMON METAMORPHIC ROCKS

Texture	Structure	
	Foliated	Massive
Coarse Crystalline	Gneiss	Metaquartzite
Medium Crystalline	Schist (Sericite) (Mica) (Talc) (Chlorite) (etc.)	Marble Quartzite Serpentine Soapstone
Fine to Microscopic	Phyllite Slate	Hornfels Anthracite coal

TABLE 11
Engineering Classification For In Situ Rock Quality

ROD %	VELOCITY INDEX	ROCK MASS QUALITY
90 - 100	0.80 - 1.00	Excellent
75 - 90	0.60 - 0.80	Good
50 - 75	0.40 - 0.60	Fair
25 - 50	0.20 - 0.40	Poor
0 - 25	0 - 0.20	Very Poor

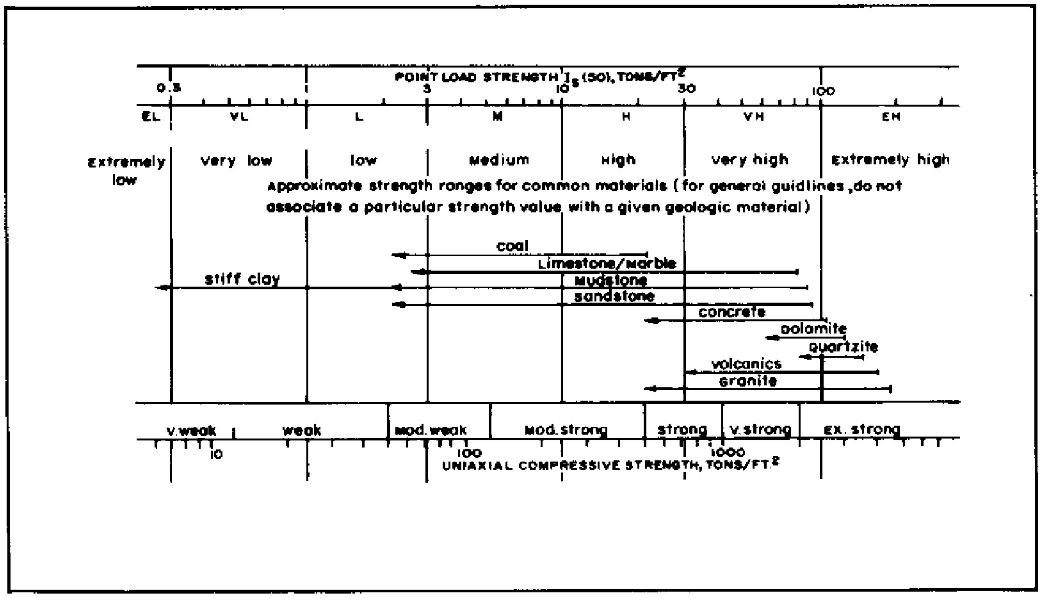


FIGURE 3
Strength Classification

load strength index is defined as the ratio of the applied force at failure to the squared distance between loaded points. This index is related to the direct tensile strength of the rock by a proportionality constant of 0.7 to 1.0 depending on the size of sample. Useful relationships of point load tensile strength index to other parameters such as specific gravity, seismic velocity, elastic modulus, and compressive strength are given in Reference 11, Prediction of Compressive Strength from Other Rock Properties, by DiAndrea, et al. The technique for performing the test is described in Reference 9.

c. Classification by Durability. Short-term weathering of rocks, particularly shales and mudstones, can have a considerable effect on their engineering performance. The weatherability of these materials is extremely variable, and rocks that are likely to degrade on exposure should be further characterized by use of tests for durability under standard drying and wetting cycle (see Reference 12, Logging Mechanical Character of Rock, by Franklin, et al.). If, for example, wetting and drying cycles reduce shale to grain size, then rapid slaking and erosion in the field is probable when rock is exposed (see Reference 13, Classification and Identification of Shales, by Underwood).

3. ENGINEERING AND PHYSICAL PROPERTIES OF ROCK. A preliminary estimate of the physical and engineering properties can be made based on the classification criteria given together with published charts, tables and correlations interpreted by experienced engineering geologists. (See Reference 8; Reference 13; Reference 14, Slope Stability in Residual Soils, by Deere and Patton; Reference 15, Geological Considerations, by Deere; Reference 16, Engineering Properties of Rocks, by Farmer.) Guidance is provided in Reference 14 for description of weathered igneous and metamorphic rock (residual soil, transition from residual to saprolite, etc.) in terms of RQD, percent core recovery, relative permeability and strength. Typical strength parameters for weathered igneous and metamorphic rocks are also given in Reference 14. Guidance on physical properties of some shales is given in Reference 13.

Section 6. SPECIAL MATERIALS

1. GENERAL CLASSIFICATION AND TYPICAL ENGINEERING IMPLICATIONS. See Table 12 for general classification and typical engineering implications of special materials that influence foundation design.

2. EXPANSIVE SOILS.

a. Characteristics. Expansive soils are distinguished by their potential for great volume increase upon access to moisture. Soils exhibiting such behavior are mostly montmorillonite clays and clay shales.

b. Identification and Classification. Figure 4 (Reference 17, Shallow Foundations, by the Canadian Geotechnical Society) shows a method based on Atterberg limits and grain size for classifying expansive soils. Activity of clay is defined as the ratio of plasticity index and the percent by weight finer than two microns (2[mu]). The swell test in a one dimensional consolidation test (see Chapter 3) or the Double Consolidometer Test (Reference 18, The Additional Settlement of Foundations Due to Collapse of Structures of

TABLE 12

Identification and Characteristics of Special Materials

Material	Geographic/Geomorphic Features	Engineering Conditions
"Quick Clay"	• Marine or brackish water clay composed of glacial rock flour that is elevated above sea level. • Generally confined to far north areas; Eastern Canada, Alaska, Scandinavia.	• Severe loss of strength when disturbed by construction activities or seismic ground shaking. • Replacement of formation water containing dissolved salt with fresh water results in strength loss. • Produces landslide prone areas (Anchorage, Alaska).
Hydraulic Fills	• Coastal facilities, levees, dikes, tailings dams	• High void ratio • Uniform gradation but variable grain sizes within same fill • High liquefaction potential • Lateral spreading • Easily eroded
Collapsing Soil	• Desert arid and semi-arid environment • Alluvial valleys, playas, loess	• Loss of strength when wetted • Differential settlement • Low density • Moisture sensitive • Gypsum/Anhydrite often present

TABLE 12 (continued)
Identification and Characteristics of Special Materials

Material	Geographic/Geomorphic Features	Engineering Conditions
Submarine Soils	• Continental shelf deposits at water depths up to several hundred feet. • Submarine canyons, turbidity flows, deltaic deposits, abyssal plain	• Distribution and physical properties of sand, silt, and clay may change with time and local geologic conditions. • Shelf deposits have few unique characteristics requiring modification of soil mechanics principals. • Local areas, such as the Gulf of Mexico have weak, underconsolidated deposits. • Deep sea calcareous deposits have water contents up to 100% and shear strengths up to about 220 psf. • Deep sea silty clays have average water contents of 100-200% and shear strengths of 35-75 psf. • Deep sea deposits are normally consolidated but near shelf deposits may be underconsolidated.
Lateritic Soils	• Tropical rainforest and savanna • Deep residual soil profile • Shield and sedimentary cover outside shield in South and Central America, Central and West Africa, southeast Asia, and other parts of the world.	• Loss of soil strength with time • High void ratio/permeability • Aggregate deterioration • Variable moisture content • Shrinkage cracks

TABLE 12 (continued)
Identification and Characteristics of Special Materials

Material	Geographic/Geomorphic Features	Engineering Conditions
Lateritic Soils (cont'd)		• Easily compacts • Shear characteristics somewhere between sand and silt • Landslide prone • Depth of wetting affects slope stability • Varied foundation conditions
Limestone and Coral	• Humid tropics and subtropics, island environment. • Karst topography accelerated in humid climates. • Limestone that are cavernous or prone to cavity formations are widely distributed throughout the world in countries of arid and humid climates. In the U. S., cavernous limestone is found in Kentucky, Pennsylvania, California, Indiana, Michigan, New Mexico, Texas, and Virginia.	• Solution cavities • Extreme variations in porosity • Void ratios in coral up to 2 • Chimney-like sinkholes and collapse structures • Slump failures, ravelling • Rock settlement and consolidation • Piles or bridging often required

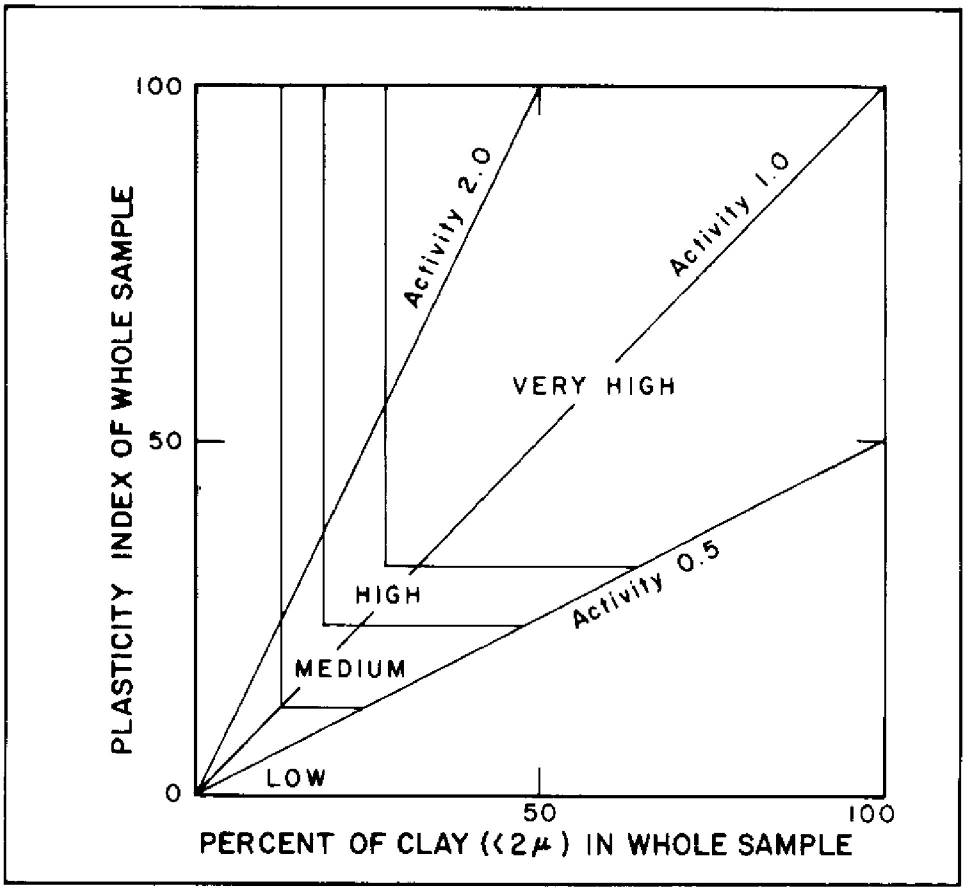


FIGURE 4
Volume Change Potential Classification for Clay Soils

Sandy Soils on Wetting, by Jennings and Knight) is used for estimating the swell potential.

3. COLLAPSING SOILS

a. Characteristics. Collapsing soils are distinguished by their potential to undergo large decrease in volume upon increase in moisture content even without increase in external loads. Examples of soils exhibiting this behavior are loess, weakly cemented sands and silts where cementing agent is soluble (e.g., soluble gypsum, halite, etc.) and certain granite residual soils. A common feature of collapsible soils is the loose bulky grains held together by capillary stresses. Deposits of collapsible soils are usually associated with regions of moisture deficiency.

b. Identification and Classification. Detailed geologic studies could identify potentially collapsible soils. Figure 5 (Reference 19, Research Related to Soil Problems of the Arid Western United States, by Holtz and Gibbs) provides guidance for identifying the potential for collapse for clayey sands and sandy clays found in the western United States. For cemented soils and nonplastic soils, criteria based on consolidometer tests are more applicable as illustrated in Figure 6 (Reference 20, A Guide to Construction on or with Materials Exibiting Additional Settlements Due to Collapse of Grain Structure, by Jennings and Knight; and Reference 21, The Origin and Occurrence of Collapsing Soil, by Knight). The potential for collapse is also evaluated in the field by performing standard plate load tests (ASTM D1194, Bearing Capacity of Soil for Static Load on Spread Footings) under varied moisture environments. For further guidance see Reference 22, Experience with Collapsible Soil in the Southwest, by Beckwith.

4. PERMAFROST AND FROST PENETRATION.

a. Characteristics. In non-frost susceptible soil, volume increase is typically 4% (porosity 40%, water volume increase in turning to ice = 10%, total heave = 40% x 10% = 4%). In susceptible soil heave is much greater as water flows to colder zones (forming ice lenses). The associated loss of support upon thaw can be more detrimental to structures than the heave itself.

b. Classification. Silts are the most susceptible to frost heave. Soils of types SM, ML, GM, SC, GC, and CL are classified as having frost heave potential.

c. Geography. Figure 7 (Reference 23, National Oceanic and Atmospheric Administration) may be used as a guide for estimating extreme depth of frost penetration in the United States.

5. LIMESTONE AND RELATED MATERIALS.

a. Characteristics. Limestone, dolomite, gypsum and anhydrite are characterized by their solubility and thus the potential for cavity presence and cavity development. Limestones are defined as those rocks composed of more than 50% carbonate minerals of which 50% or more consist of calcite and/or aragonite. Some near shore carbonate sediments (also called limestone, marl, chalk) could fit this description. Such sediments are noted for erratic degrees of induration, and thus variability in load supporting capacity and

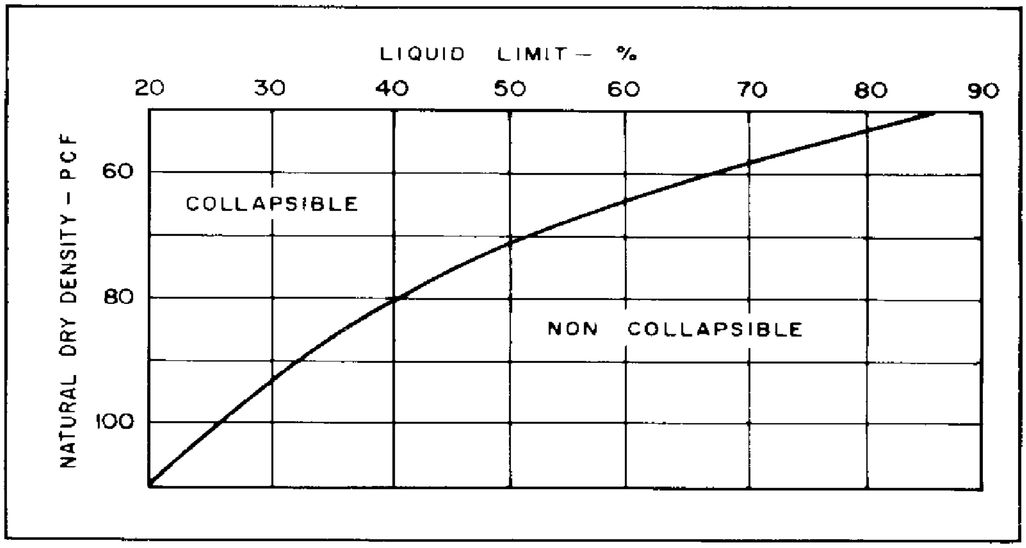
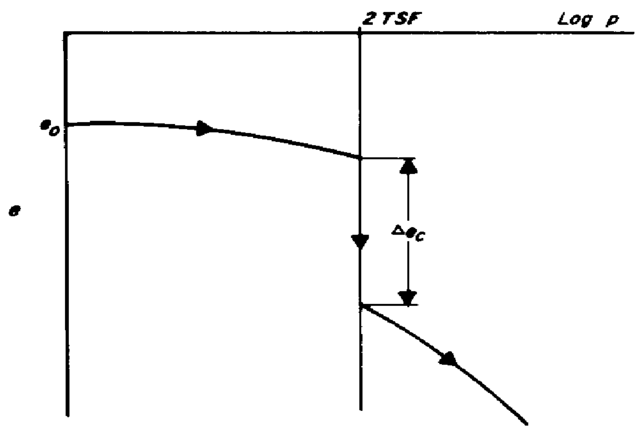


FIGURE 5
Criterion for Collapse Potential



Collapse Potential (CP) is defined as:

$$CP = \frac{\Delta e_c}{1 + e_o} \quad \text{or} \quad CP = \frac{\Delta Hc}{Ho}$$

Δe_c = Change in void ratio upon wetting

e_o = Natural void ratio

ΔHc = Change in height upon wetting

Ho = Initial height

Collapse Potential Values

CP	Severity of Problem
0-1%	No problem
1-5%	Moderate trouble
5-10%	Trouble
10-20%	Severe Trouble
20%	Very Severe Trouble

FIGURE 6
Typical Collapse Potential Test Results

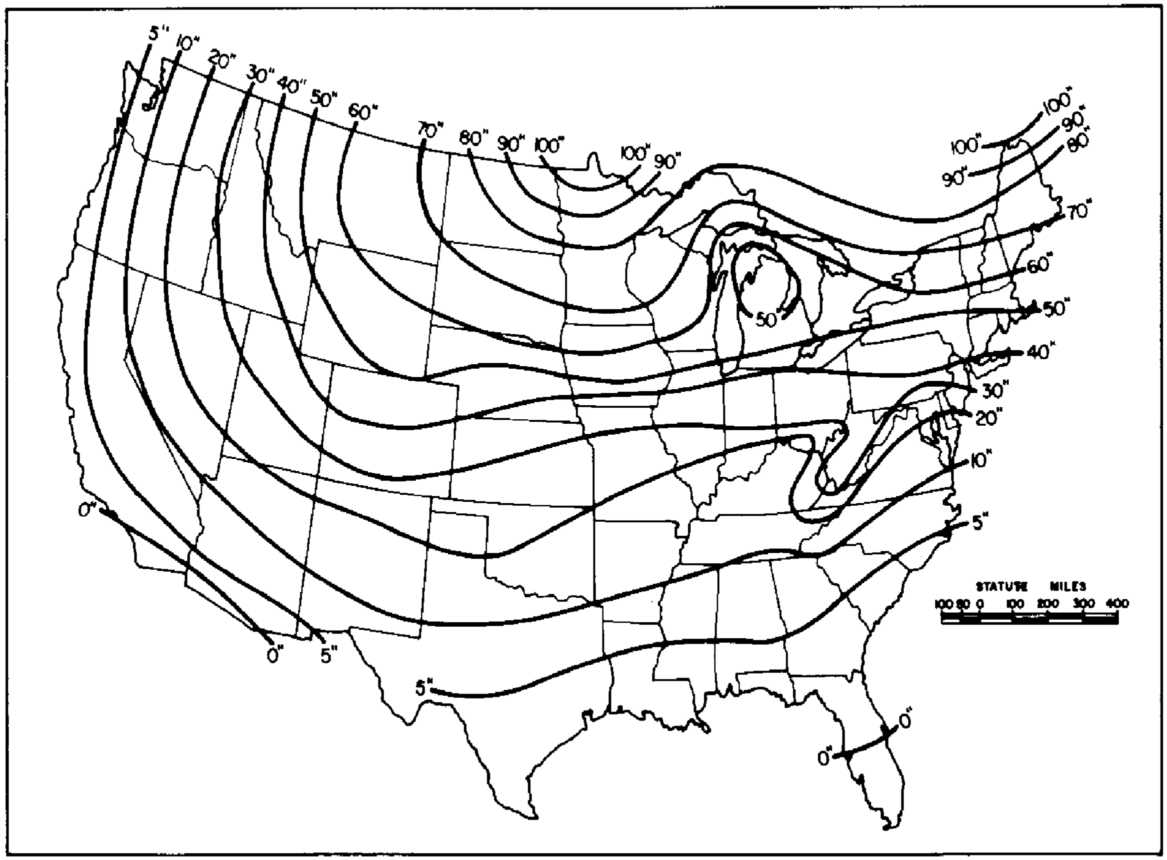


FIGURE 7
Extreme Frost Penetration (in inches) Based Upon State Average

uncertainty in their long-term performance under sustained loads. The most significant limestone feature is its solubility. An extremely soluble one can be riddled with solution caves, channels, or other open, water, or clay filled features.

b. Identification. Presence of solution features may be checked by geological reconnaissance, drilling, and other forms of bedrock verification. Geophysical techniques, including shallow seismic refraction, resistivity and gravimetry are often found to be valuable supplements.

c. Coral and Coral Formation.

(1) Origin. Living coral and coralline debris are generally found in tropical regions where the water temperature exceeds 20deg.C. Coral is a term commonly used for the group of animals which secrete an outer skeleton composed of calcium carbonate, and which generally grow in colonies. The term "coral reef" is often applied to large concentrations of such colonies which form extensive submerged tracts around tropical coasts and islands. In general, coralline soils deposited after the breakdown of the reef, typically by wave action, are thin (a few meters thick) and form a veneer upon cemented materials (limestones, sandstones, etc.).

(2) Geological Classification. Because the granular coralline and algal materials are derived from organisms which vary in size from microscopic shells to large coralheads several meters in diameter, the fragments are broadly graded and range in size from boulders to fine-grained muds. Similarly, the shape of these materials varies from sharp, irregular fragments to well-rounded particles. Coralline deposits are generally referred to as "biogenic materials" by geologists. When cemented, they may be termed "reefrock," or "beachrock," or other names which imply an origin through cementation of particles into a hard, coherent material.

(3) Characteristics. Coralline deposits are generally poor foundation materials in their natural state because of their variability and susceptibility to solution by percolating waters, and their generally brittle nature. Coralline materials are often used for compacted fill for roads and light structures. Under loads, compaction occurs as the brittle carbonate grains fracture and consolidate. They can provide a firm support for mats or spread footings bearing light loads, but it is necessary to thoroughly compact the material before using it as a supporting surface. Heavy structures in coral areas are generally supported on pile foundations because of the erratic induration. Predrilling frequently is required.

Because of extreme variability in engineering properties of natural coral formations, it is not prudent to make preliminary engineering decisions on the basis of "typical properties." Unconfined compression strengths of intact specimens may range from 50 tons/ft.2- to 300 tons/ft.2-, and porosity may range from less than 40% to over 50%.

For further guidance see Reference 24, Failure in Limestone in Humid Subtropics, by Sowers, which discusses factors influencing construction in limestone; and Reference 25, Terrain Analysis - A Guide to Site Selection Using Aerial Photographic Interpretation, by Way.

6. QUICK CLAYS.

a. Characteristics. Quick clays are characterized by their great sensitivity or strength reduction upon disturbance.

All quick clays are of marine origin. Because of their brittle nature, collapse occurs at relatively small strains. Slopes in quick clays can fail without large movements. For further guidance see Reference 5 and Reference 26, Quick Clays and California: No Quick Solutions, by Anne.

b. Identification. Quick clays are readily recognized by measured sensitivities greater than about 15 and by the distinctive, strain-softening shape of their stress-strain curves from strength or compressibility tests.

7. OTHER MATERIALS AND CONSIDERATIONS.

a. Man-Made Fills. Composition and density are the main concerns. Unless these can be shown to be non-detrimental to the performance of the foundation, bypassing with deep foundations, or removal and replacement are in order.

Sanitary landfills may undergo large settlements under self weight as well as under structural loads. Guidelines on the evaluation of settlement and other foundation considerations for sanitary landfills are given in DM-7.3, Chapter 3.

b. Chemically Reactive Soils. For foundation construction, the main concerns usually are corrosion and gas generation. Corrosion potential is determined in terms of pH, resistivity, stray current activity, groundwater position, chemical analysis, etc.; and a compatible foundation treatment, e.g., sulfate resistant concrete, lacquers, creosote, cathodic protection, etc., is prescribed. For gas concentration, organic matter content and field testing for gas are usually performed. If gas generation is expected, some form of venting system is designed (see Chapter 2). The potential presence of noxious or explosive gases should be considered during the construction excavations and tunneling.

c. Lateritic Soils. Lateritic soils are found in tropical climates throughout the world. Typical characteristics are shown in Table 12. For further guidance see Reference 27, Laterite Soil Engineering, by Gidigasu; Reference 28, Laterite Genesis, Location, Use, by Persons; Reference 29, Engineering Study of Laterite and Lateritic Soils in Connection with Construction of Roads, Highways and Airfields, by the U.S. Agency for International Development; Reference 30, Laterite, Lateritic Soils and Other Problem Soils of Africa, by the U.S. Agency for International Development; and Reference 31, Laterite and Lateritic Soils and Other Problem Soils of the Tropics, by the U.S. Agency for International Development.

d. Submarine Soils. Typical characteristics are shown in Table 12. Further guidance may be found in Reference 32, Engineering Properties of Submarine Soils: State-of-the-Art Review, by Noorany and Gizienski.

REFERENCES

1. Departments of the Army and Air Force, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), TM 5-818-1/AFM 88-3, Chapter 7, Washington, DC, 1979.

2. Mitchell, J. K. and Lunne, T. A., Cone Resistance as Measure of Sand Strength, Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT7, 1978.

3. Terzaghi, K., and Peck, R. B., Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, 1967.

4. Ayers J., and Plum, R., Unpublished work.

5. Hough, B. K., Basic Soils Engineering, Ronald Press, New York, 1969.

6. ISRM Working Party, Suggested Methods of the Description of Rock Masses Joints and Discontinuities, International Society of Rock Mechanics Second Draft of Working Party, Lisbon, 1975.

7. Geological Society of America, Rock Color Chart.

8. Coon, J. H. and Merritt, A. H., Predicting Insitu Modulus of Deformation Using Rock Quality Indexes, Determination of the Insitu Modulus of Deformation of Rock, STP 457, ASTM 1970.

9. Deere, D. U., Hendron A. J. Jr., Patton, F. D. and Cording, E. J., Design of Surface and Near Surface Construction in Rock, Proceedings, Eighth Symposium on Rock Mechanics, MN., 1966.

10. Broch, E. and Franklin, J. A., The Point Load Strength Test, International Journal of Rock Mechanics and Mining Science, Pergamon Press, Vol. 9, pp 669 - 697, 1972.

11. DiAndrea, D. V., Fischer, R. L., and Fogelson, D. E., Prediction of Compressive Strength from Other Rock Properties, U. S. Bureau of Mines, Report Investigation 6702, p 23, 1967.

12. Franklin, J. A., Broch, E., and Walton, G., Logging Mechanical Character of Rock, Transactions, Institution of Mining and Metallurgy, A 80, A1-A9, 1971.

13. Underwood, L. B., Classification and Identification of Shales, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM6, 1962.

14. Deere, D. U. and Patton, F. D., Slope Stability in Residual Soils, Proceedings of the Fourth Panamerican Conference on Soil Mechanics and Foundation Engineering, San Juan, Volume 1, pp 87-100, 1971.

15. Deere, D. U., Geologic Considerations, Rock Mechanics in Engineering Practice, Stagg, K. G. and Zienkiewics, O. C., ed., John Wiley and Sons, New York, Chapter 1, 1969.

16. Farmer, I. W., Engineering Properties of Rocks, E & FN Spon LTP, London, 1968.

17. The Canadian Geotechnical Society, Shallow Foundations, Part 2, Canadian Foundation Engineering Manual, 1978.

18. Jennings, J. E. and Knight, K., The Additional Settlement of Foundations Due to Collapse of Structures of Sandy Soils on Wetting, Proceeding of the Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

19. Holtz, W. G. and Gibbs, H. J., Research Related to Soil Problems of the Arid Western United States, Proceedings of the Third Panamerican Conference on Soil Mechanics and Foundation Engineering, Caracas, 1967.

20. Jennings, J. E. and Knight, K., A Guide to Construction on or With Materials Exhibiting Additional Settlements Due to Collapse of Grain Structure, Proceedings of the Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, pp 99-105, 1975.

21. Knight, K., The Origin and Occurrence of Collapsing Soil, Proceedings of the Third Regional Conference of Africa on Soil Mechanics and Foundation Engineering, Vol. 1, pp 127-130, 1963.

22. Beckwith, G. H., Experience with Collapsible Soil in the Southwest, ASCE Conference, Arizona Section, 1979.

23. National Oceanic and Atmospheric Administration, Environmental Data and Information Service, Asheville, NC.

24. Sowers, F. G., Failure in Limestone in Humid Subtropics, Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT8, 1975.

25. Way, S. D., Terrain Analysis - A Guide to Site Selection Using Aerial Photographic Interpretation, Dowden, Hutchinson & Ross, Inc., Stroudsburg, PA., 1973.

26. Anne, Q. A., Quick Clays and California: No Quick Solutions, Focus on Environmental Geology, Ronald Rark, ed., pp 140-145, 1973.

27. Gidigasu, M. D., Laterite Soil Engineering, Elsevier Scientific Publishing Co., 1976.

28. Persons, S. B., Laterite Genesis, Location, Use, Plenum Press, 1970.

29. U.S. Agency for International Development, Engineering Study of Laterite and Lateritic Soils in Connection With Construction of Roads, Highways and Airfields, Southeast Asia, 1969.

30. U.S. Agency for International Development, Laterite, Lateritic Soils and Other Problem Soils of Africa, 1971.

31. U.S. Agency for International Development, Laterite and Lateritic Soils and Other Problem Soils of the Tropics, 1975.

32. Noorany, I. and Gizienski, S. F., Engineering Properties of Submarine Soils: State-of-the-Art Review, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM5, 1970.

33. Naval Facilities Engineering Command, Design Manuals (DM)

DM-5.04	Pavements
DM-21 series	Airfield Pavement

Government agencies may obtain copies of design manuals from the U. S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, Pennsylvania 19120. Nongovernment agencies and commercial firms may obtain copies from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

CHAPTER 2. FIELD EXPLORATION, TESTING, AND INSTRUMENTATION

Section 1. INTRODUCTION

1. SCOPE. This chapter contains information on exploration methods including use of air photos and remote sensing, geophysical methods, test pits, test borings, and penetrometers. Also presented is information on methods of sampling, measuring in situ properties of soil and rock, field measurements, and geotechnical monitoring equipment.

2. RELATED CRITERIA. For other criterial related to exploration and sampling, see the following sources:

Subject	Sources
Soil Exploration and Subgrade Testing	NAVFAC DM-5.04
Field Pumping Tests	NAVFAC P-418

3. PLANNING FOR FIELD INVESTIGATIONS. The initial phase of field investigations should consist of detailed review of geological conditions at the site and in its general environs. This should include a desk top study of available data including remote sensing imagery, aerial photography, and a field reconnaissance. The information obtained should be used as a guide in planning the exploration.

To the extent possible, borings should be supplemented by lower cost exploration techniques such as test pits, probes, seismic refraction surveys, and electrical resistivity surveys. This is particularly true in the offshore environment where borings are exceptionally expensive.

Information on boring layout is given in Section 5 and a sample boring log is given in Figure 1. Guidance on exploration techniques is given in Sections 5 and 6.

It should be noted that NAVFAC has a Geotechnical Data Retrieval System. To optimize its use, the U.S. Navy encourages utilization of its format on Navy projects. Details relative to this can be found in Reference 1, Geotechnical Data Retrieval System, by NAVFAC.

4. EXPLORATION PHASES. Project exploration can generally have three phases: reconnaissance/feasibility exploration; preliminary exploration; and detailed/final exploration. Additional exploration may be required during or after construction. Frequently, all preconstruction phases are combined into a single exploration effort.

a. Reconnaissance/Feasibility. Reconnaissance includes a review of available topographic and geologic information, aerial photographs, data from previous investigations, and site examination. Geophysical methods are applicable in special cases. Reconnaissance/feasibility frequently reveals difficulties which may be expected in later exploration phases and assists in determining the type, number and locations of borings required.

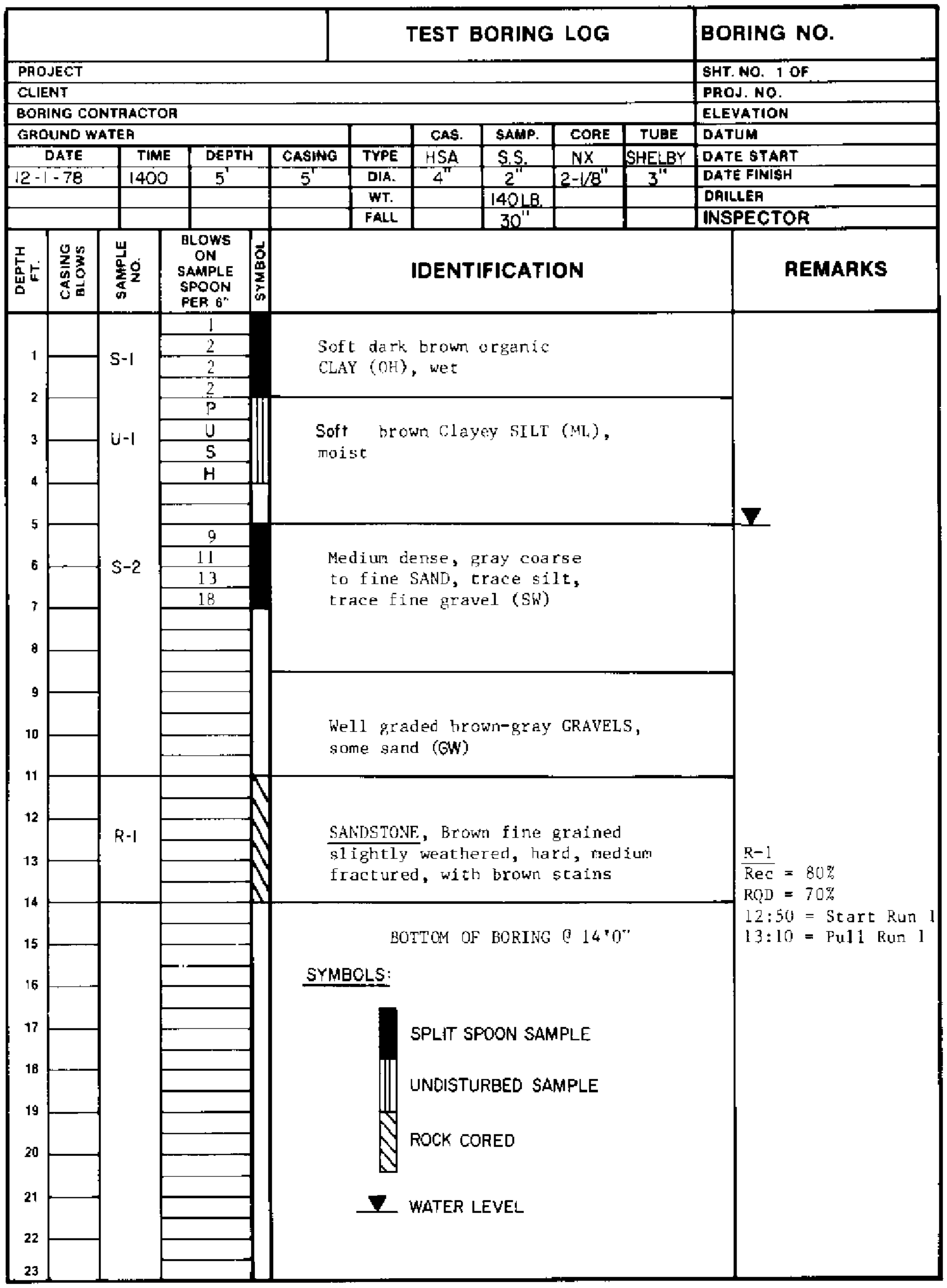


FIGURE 1
Sample Boring Log

b. Preliminary Exploration. This may include borings to recover samples for identification tests only.

c. Detailed Exploration. This phase normally includes borings, disturbed and undisturbed sampling for laboratory testing, standard penetration resistances, and other in situ measurements. At critical sites it may also include test pits, piezometer measurements, pumping tests, etc.

d. Construction/Post Construction Phases. Further evaluation of foundation conditions may be required during the construction phase. Monitoring of the site or structure may be necessary throughout the construction and post construction phases.

Section 2. PUBLISHED SOIL AND GEOLOGICAL MAPS

1. SOURCES. Data on the physical geology of the United States are available in maps and reports by government agencies, universities, and professional societies (see Table 1). These sources often contain geological information on foreign countries.

2. PREVIOUS INVESTIGATIONS. For studies in developed areas, collect information from previous work on foundations and subsurface conditions.

a. Shipyard or Waterfront Areas. These locations often have undergone cycles of expansion and reconstruction with older foundations remaining buried in place. Records of former construction may contain information on borings, field tests, groundwater conditions, and potential or actual sources of trouble.

b. Evaluation. Review of data from previous work should receive the greatest attention of any phase in a reconnaissance investigation.

Section 3. REMOTE SENSING DATA METHODS

1. SOURCES. Remote sensing data are acquired by imagery recovery devices and their transporting media. Aerial photographs are the most common type with coverage of almost the entire United States available at scales from 1:12,000 to 1:80,000. With the advent of improving technology, space programs and data gathering satellites, a wealth of other remote sensed data are now available for use. Table 2 summarizes the types of data most commonly used in engineering studies. Photos at larger scale up to 1:2000 are available for some locations from state agencies and commercial aero-photogrammetric firms.

2. UTILIZATION. Use of photographs and mosaics is routine in most large engineering studies such as highway and airfield work. Other forms of remote sensing data are used on a more selective basis when required. For a complete description on the use of imagery in earthquake analysis, see Reference 2, Imagery in Earthquake Analysis, by Glass and Slemmons. For unfamiliar sites, the air photographs aid in planning and layout of an appropriate boring program.

TABLE 1
Sources of Geological Information

Series	Description of Material
U.S. Geological Survey (USGS)	Consult USGS Index of Publications from Superintendent of Documents, Washington, D.C. Order publications from Superintendent of Documents. Order maps from USGS, Washington, D.C. Contact regional distribution offices or information.
Geological index map	Individual maps of each state showing coverage and sources of all published geological maps.
Folios of the Geological Atlas of the United States	Contains maps of bedrock and surface materials for many important urban and seacoast areas. When out of print, obtain folios through suppliers of used technical literature.
Geological Quadrangle Maps of United States	This series supplants the older geological folios including areal or bedrock geology maps with brief descriptive text. Series is being extended to cover areas not previously investigated.
Bulletins, professional papers, circulars, annual reports, monographs	General physical geology emphasizing all aspects of earth sciences, including mineral and petroleum resources, hydrology and seismicity. Areal and bedrock geology maps for specific locations included in many publications.
Water supply papers	Series includes papers on groundwater resources in specific localities and are generally accompanied by description of subsurface conditions affecting groundwater plus observations of groundwater levels.
Topographic maps	Topographic contour maps in all states, widespread coverage being continually expanded.
Libraries	Regional office libraries contain geological and seismological information from many sources. Data on foreign countries are often suitable.

TABLE 1 (continued)
Sources of Geological Information

Series	Description of Material
National Oceanic and Atmospheric Administration (NOAA), National Ocean Survey (NOS)	Consult Catalog 1, Atlantic and Gulf Coasts; 2, Pacific Coast, 3, Alaska; 4, Great Lakes; and 5, Bathymetric Maps and Special Charts. Order from Distribution Service, National Ocean Survey, Riverdale, Maryland 20840
Nautical Charts	Charts of coastal and inland waterways showing available soundings of bottom plus topographic and cultural features adjacent to the coast or waterways.
U.S. Department of Agriculture (USDA), Soil Conservation Service.	Consult "List of Published Soil Surveys," USDA, Soil Conservation Service, January 1980 (published annually). Listing by states and countries.
Soil maps and reports	Surveys of surface soils described in agricultural terms. Physical geology summarized. Excellent for highway or airfield investigations. Coverage mainly in midwest, east, and southern United States.
State Geological Surveys/State Geologist's Office	Most states provide excellent detailed local geological and reports covering specific areas or features in the maps publications of the state geologists. Some offices are excellent sources of information on foreign countries.
Geological Society of America (GSA)	Write for index to GSA, P. O. Box 9140, 3300 Penrose Place, Boulder, Colorado, 80302.
Monthly bulletins, special papers, and memoirs.	Texts cover specialized geological subjects and intensive investigations of local geology. Detailed geological maps are frequently included in the individual articles.
Geological maps	Publications include general geological maps of North and South America, maps of glacial deposits, and Pleistocene aeolian deposits.

TABLE 1 (continued)
Sources of Geological Information

Series	Description of Material
Library of Congress	Maintains extensive library of U.S. and foreign geologic reports by geographical area. Inquiry to Library of Congress, 10 First Street, Washington, D. C., 20540.
Worldwide National Earth-Science Agencies	For addresses consult "Worldwide Directory of National Earth-Science Agencies," USGS Circular 716, 1975

TABLE 2
Remote Sensing Data

TYPE	DESCRIPTION AND GENERAL USE	AVAILABILITY
Aerial Photography	Available in 9-inch frames with overlap for stereoscopic viewing. Valuable because of high resolution and available scales could range from 1:12,000 (or larger) to 1:80,000. Photos used extensively for topographic and/or geologic mapping, drainage patterns, and other uses include identifying location of existing structures, vegetation, access routes and site locations for planned explorations.	U.S. Geological Survey (USGS); National Information Center (NCIC), Reston, VA, U.S. Soil Conservation Service (SCS); U.S. Forest Service; U.S. Bureau of Land Management; Tennessee Valley Authority.
	Imagery obtained by satellite which flies in circular orbit 570 miles above Earth's surface and circles Earth about 14 times a day. Gives repetitive coverage every 18 days. The primary sensor is the multispectral scanner (MSS) which acquires images 115 miles per side in four spectral bands. The four bands are: BAND 4: The green band, 0.5 to 0.6 micrometers, emphasizes movement of sediment-laden water and delineates areas of shallow water, such as shoals, reefs, etc., useful in differentiating lithology;	From Earth Resources Observation System (EROS) Data Center, Sioux Falls, SD 57198. Closest regional source can be determined by calling (605) 594-6511, Ext. 151. Imagery available in scales of 1:1,000,000; 1:400,000; and 1:250,000. 1978 prices ranged from \$8.00 for black and white images at 1:1,000,000 to \$50.00 for color infrared composite at 1:250,000.

TABLE 2 (continued)
Remote Sensing Data

TYPE	DESCRIPTION AND GENERAL USE	AVAILABILITY
	BAND 6: The red band, 0.6 to 0.7 micrometers, emphasizes cultural features, such as metropolitan areas; BAND 7: The near-infrared band, 0.7 to 0.8 micrometers, emphasizes vegetation, the boundary between land and water, landforms and useful in structural interpretation of geology; BAND 8: The second near-infrared band 0.8 to 1.1 micrometers, provides the best penetration of atmospheric haze, the best band for detecting faults, lineaments, mega-joint patterns or other structural features, and also emphasizes vegetation, the boundary between land and water, and landforms.	
Skylab	Satellite orbit 270 miles above earth with system which includes a six lens multi-spectra camera and an Earth terrain camera. Six lens array designed to provide high-quality photography of Earth's surface. Films used were filtered black and white, color and false color infrared. Area covered by	From EROS Data Center, Sioux Falls, SD 47198. Photos can be enlarged to scale of 1:250,000 with almost no loss of information.

TABLE 2 (continued)
Remote Sensing Data

TYPE	DESCRIPTION AND GENERAL USE	AVAILABILITY
Skylab (cont'd)	each image is 100 x 100 miles. The Earth terrain camera provided high resolution photography for scientific study. Various black and white, color and false-color infrared films used. Each frame covers 70 x 70 miles. Limited data were acquired between latitudes 40 degrees north and 50 degrees south in 1973-74 flights. Skylab flights are completed. Photography is useful for regional planning, environmental studies, and geologic analyses.	
NASA	Aerial photography produced from NASA Earth Resources Aircraft Program. Photos available in wide variety of formats from flights as low as a few thousand feet to U-2 flights at altitudes above 60,000 feet. High altitude photos generally available at scales of about 1:120,000 and 1:60,000. At 1:120,000 scale area covered is about 17 miles on a side. Photos available in black and white, color, or false-color infrared. Coverage not available for all areas. Flights provide good resolution photos for planning, environmental studies or site oriented studies; color IR excellent for fault/lineament evaluation.	Purchase from EROS Data Center, Sioux Falls, SD 57198. Prices in 1978 range upward from $8.00 for 1:120,000 scale black and white photos.

TABLE 2 (continued)
Remote Sensing Data

TYPE	DESCRIPTION AND GENERAL USE	AVAILABILITY
SLAR	Side-looking airborne radar (SLAR) is especially applicable in areas of persistent cloud cover and can be essentially obtained in all-weather, day-night operations. Radar uses low, oblique illumination angles giving appearance of low sun angle imagery. It gives large area views of Earth's surface being available in scales ranging from 1:2,000,000 to 1:250,000. SLAR should not be used to replace air photos; it is a valuable complement to photos for regional studies. This is the best imagery for regional structural (faults/lineaments) analysis, often increasing detection of lineaments by 100-200%.	National Cartographic Information Center (NCIC), Reston, VA.; Goodyear Aerospace Corporation and Motorola, Litchfield Park, AZ.; Westinghouse Electric Corp., Philadelphia, PA.
Thermal IR	Thermal infrared sensors detect the different intensity of infrared emission (or heat) from an object or the Earth surface. Where temperature contrasts are significant, thermal IR imagery can be useful. Ordinarily it is used for special purposes or projects and could be useful as a complement to other remote sensing data during a planning and siting study. Useful in fault detection in covered alluvial areas, geothermal exploration, location of seepage, location of near surface peat deposits, covered meander scars, and heat loss studies.	Very little of Earth's surface covered. Mostly obtained as needed; most aerial survey firms have capability of flying thermal IR at prices comparable to large scale photographic coverage. A recent satellite, Heat Capacity Mapping Mission (HCMM) has been acquiring thermal IR data over the U.S. and portions of foreign countries. Hard copy images to eventually be available through National Space Science Data Center, Goddard Space Flight Center, Greenbelt, MD. No cost data available.

a. Flight strips. Most aerial photographs are taken as flight strips with 60 percent or more overlap between pictures along the flight line and 20 to 30 percent side overlap between parallel flight lines.

b. Interpretation. When overlapping pictures are viewed stereoscopically, ground relief appears. From the appearance of land forms or erosional or depositional features, the character of soil or rock may be interpreted (see Reference 3, Terrain Analysis, A Guide to Site Selection Using Aerial Photographic Interpretation, by Way, for guidance on interpretation and terrain analysis with respect to issues in site development).

3. LIMITATIONS. Interpretation of aerial photographs and other remote sensed data requires considerable experience and skill, and results obtained depend on the interpreter's proficiency. Spot checking in the field is an essential element in photo-geologic interpretation.

a. Accuracy. Accuracy is limited where dense vegetation obscures ground features (unless SLAR imagery is used) and is dependent upon the scale, sensors, film products and enlargements. Recently, computer enhancements of multi-spectral imagery has made LANDSAT data compatible with conventional aerial photography.

b. Utility. For intensive investigations within developed areas, aerial photographs are not essential to exploration. Although valuable, the technique does not provide quantitative information for site specific foundation conditions. However, photo-interpretation greatly aids qualitative correlation between areas of known and unknown subsurface conditions.

Section 4. GEOPHYSICAL METHODS

1. UTILIZATION. See Table 3 for onshore and Table 4 for offshore geophysical methods and application.

a. Advantages. In contrast to borings, geophysical surveys explore large areas rapidly and economically. They indicate average conditions along an alignment or in an area rather than along the restricted vertical line at a single location as in a boring. This helps detect irregularities of bedrock surface and interface between strata.

b. Applications. Geophysical methods are best suited to prospecting sites for dams, reservoirs, tunnels, highways, and large groups of structures, either on or offshore. They also have been used to locate gravel deposits and sources of other construction materials where properties differ substantially from adjacent soils. Downhole, uphole and cross-hole seismic surveys are used extensively for determining dynamic properties of soil and rock at small strains.

(1) Rippability-velocity relationships for various rock types are given in DM-7.2 Chapter 1.

TABLE 2
Remote Sensing Data

TYPE	DESCRIPTION AND GENERAL USE	AVAILABILITY
Aerial Photography	Available in 9-inch frames with overlap for stereoscopic viewing. Valuable because of high resolution and available scales could range from 1:12,000 (or larger) to 1:80,000. Photos used extensively for topographic and/or geologic mapping, drainage patterns, and other uses include identifying location of existing structures, vegetation, access routes and site locations for planned explorations.	U.S. Geological Survey (USGS); National Information Center (NCIC), Reston, VA, U.S. Soil Conservation Service (SCS); U.S. Forest Service; U.S. Bureau of Land Management; Tennessee Valley Authority.
	Imagery obtained by satellite which flies in circular orbit 570 miles above Earth's surface and circles Earth about 14 times a day. Gives repetitive coverage every 18 days. The primary sensor is the multispectral scanner (MSS) which acquires images 115 miles per side in four spectral bands. The four bands are: BAND 4: The green band, 0.5 to 0.6 micrometers, emphasizes movement of sediment-laden water and delineates areas of shallow water, such as shoals, reefs, etc., useful in differentiating lithology;	From Earth Resources Observation System (EROS) Data Center, Sioux Falls, SD 57198. Closest regional source can be determined by calling (605) 594-6511, Ext. 151. Imagery available in scales of 1:1,000,000; 1:400,000; and 1:250,000. 1978 prices ranged from $8.00 for black and white images at 1:1,000,000 to $50.00 for color infrared composite at 1:250,000.

TABLE 3 (continued)
Onshore Geophysics for Engineering Purposes

Name of Method	Procedure or Principle Utilized	Applicability and Limitations
Uphole, Downhole and Cross-hole Surveys	Uphole or downhole: Geophones on surface, energy source in borehole at various locations starting from hole bottom. Procedure can be revised with energy source on surface, detectors moved up or down the hole. Downhole: Energy source at the surface (e.g., wooden plank struck by hammer), geophone probe in borehole. Cross-hole: Energy source in central hole, detectors in surrounding holes.	Obtain dynamic soil properties at very small strains, rock mass quality, cavity detection. Unreliable for irregular strata or soft strata with large gravel content. Also unreliable for velocities decreasing with depth. Cross-hole measurements best suited for in situ modulus determination.
ELECTRICAL METHODS Resistivity	Based on the difference in electrical conductivity or resistivity of strata, resistivity of subsoils at various depths is determined by measuring the potential drop and current flowing between two current and two potential electrodes from a battery source. Resistivity is correlated to material type.	Used to determine horizontal extent and depths up to 100 feet of subsurface strata. Principal applications for investigating foundations of dams and other large structures, particularly in exploring granular river channel deposits or bedrock surfaces. Also used for locating fresh/salt water boundaries.
Drop in Potential	Based on the determination of the ratio of potential drops between 3 potential electrodes as a function of the current imposed on 2 current electrodes.	Similar to resistivity methods but gives sharper indication of vertical or steeply inclined boundaries and more accurate depth determinations. More susceptible than resistivity method to surface interference and minor irregularities in surface soils.

TABLE 3 (continued)
Onshore Geophysics for Engineering Purposes

Name of Method	Procedure or Principle Utilized	Applicability and Limitations
E-Logs	Based on differences in resistivity and conductivity measured in borings as the probe is lowered or raised.	Useful in correlating units between borings, has been used to correlate materials having similar seismic velocities. Generally not suited to civil engineering exploration but valuable in geologic investigations.
MAGNETIC MEASUREMENTS	Highly sensitive proton magnetometer is used to measure the Earth's magnetic field at closely spaced stations along a traverse.	Difficult to interpret in quantitative terms but indicates the outline of faults, bedrock, buried utilities, or metallic trash in fills.
GRAVITY MEASUREMENTS	Based on differences in density of subsurface materials as indicated by the vertical intensity or the curvature and gravitational field at various points being investigated.	Useful in tracing boundaries of steeply inclined subsurface irregularities such as faults, intrusions, or domes. Methods not suitable for shallow depth determination but useful in regional studies. Some application in locating limestone caverns.

TABLE 4
Offshore Geophysical Methods

Equipment	Purpose	Characteristics	Capabilities
Depth Recorders:			
Fathometer	Precision depth recording determining bathymetry.	Most recording sounders operate 200 KHz, pipe mounted transducer. Little subbottom penetration.	Four depth ranges cover 0-250 feet; range doubling switch permits bottom tracking to 410 feet; accuracy of 0.5% of indicated depth.
Seismic Reflection Profilers:			
Stratasonde Acoustic Hypacs	Seismic profiling (shallow) - characteristics of surface materials.	Low-frequency SONAR-type transducer profiling system; operates at 3.5 and 7 KHz frequency; high resolution due to short pulse length and high repetition rate.	Resolve reflecting layers within 3-4 feet of the bottom; penetration capabilities of 50 feet or less.
Acoustipulse Boomer	Seismic profiling (intermediate) - characteristics of surface and subsurface materials.	Electromechanical transducer; short duration, high power electrical pulse discharges from an energy source into an electromagnetic coil controlled metal plate, generating a repeatable sound pulse; mounted in a catamaran sled towed by vessel; board band acoustic pulse in 500-800 Hz region.	Operates in water depth from 10-600 feet; provides moderate resolution with moderate penetration up to 300 feet or more for geologic and engineering investigation.

TABLE 4 (continued)
Offshore Geophysical Methods

Equipment	Purpose	Characteristics	Capabilities
Sparker	Seismic profiling (deep) - geologic structure of bedrock.	Low-frequency, high energy sound generated by rapid discharge of electrical energy between electrodes and a surrounding frame; a plasma bubble is formed in the frequency range of 100-500 Hz and energy discharges 100-3000 joules.	Operates in water depths of 40-2000 feet, resolution capabilities of 50-80 feet with penetration depths of hundreds to thousands of feet depending upon energy selection.
Side Scan Sonar	Bottom surface features.	Mark 1B; SONAR image of ocean bottom up to 500 meters on each side of tow fish; operates at 105 KHz frequency; new safety release harness allows recovery of tow fish when obstruction is encountered; acoustic reflectors, (rocks, metal objects, sand ripples) are shown by dark areas; depressions are shown by light areas.	High resolution scanning can differentiate various bottom materials, locate hazards or obstructions (submerged hulks, outcrops).

c. Criteria. No definite criteria for geophysical methods can be given because they are highly specialized and require experienced operators and interpreters for each application.

2. LIMITATIONS. Geophysical surveys are able to outline boundaries between strata, but can only indicate approximate soil properties.

a. Sources of Errors. Differences in degree of saturation, presence of mineral salts in groundwater, or similarities of strata that effect transmission of seismic waves may lead to vague or distorted conclusions.

b. Check Borings. Geophysical surveys should be supplemented by borings and sampling to determine soil properties and confirm the stratification revealed by the survey.

Section 5. SOIL BORINGS AND TEST PITS

1. SOIL BORINGS. Soil borings are probably the most common method of subsurface exploration in the field.

a. Boring Methods. See Table 5 for applicability of the several methods of making soil borings. For details of boring techniques and equipment, see Reference 4, Subsurface Exploration and Sampling for Civil Engineering Purposes, by Hvorslev.

b. Boring Layout. General guidance for preliminary and final boring layout is presented in Table 6 according to the type of structure or problem being investigated. Boring layout should also be governed by the geology of the site.

(1) Geological Sections. Arrange borings so that geological sections may be determined at the most useful orientations for final siting and design. Borings in slide areas should establish the full geological section necessary for stability analyses.

(2) Critical Strata. Where detailed settlement, stability, or seepage analyses are required, include a minimum of two borings to obtain undisturbed samples of critical strata. Provide sufficient preliminary sample borings to determine the most representative location for undisturbed sample borings.

c. Boring Depths. The depth to which borings should be made depends on the sizes and types of proposed structures (see Table 7). It is also controlled to a great degree by the characteristics and sequence of the subsurface materials encountered.

(1) Unsuitable Foundation Strata. Extend all borings through unsuitable foundation strata, such as unconsolidated fill; peat; highly organic materials; soft, fine-grained soils; and loose, coarse-grained soils to reach hard or compact materials of suitable bearing capacity.

TABLE 5
Types of Test Borings

Boring Method	Procedure Utilized	Applicability
Auger boring	Hand or power operated augering with periodic removal of material. In some cases continuous auger may be used requiring only one withdrawal. Changes indicated by examination of material removed. Casing generally not used.	Ordinarily used for shallow explorations above water table in partly saturated sands and silts, and soft to stiff cohesive soils. May be used to clean out hole between drive samples. Very fast when power-driven. Large diameter bucket auger permits examination of hole. Hole collapses in soft soils and soils below groundwater table.
Hollow-stem flight auger	Power operated, hollow stem serves as a casing.	Access for sampling (disturbed or undisturbed) or coring through hollow stem. Should not be used with plug in granular soil. Not suitable for undisturbed sampling in sand and silt.
Wash-type boring for undisturbed or dry sample	Chopping, twisting, and jetting action of a light bit as circulating drilling fluid removes cuttings from holes. Changes indicated by rate of progress, action of rods, and examination of cuttings in drilling fluid. Casing used as required to prevent caving.	Used in sands, sand and gravel without boulders, and soft to hard cohesive soils. Most common method of subsoil exploration. Usually can be adapted for inaccessible locations, such as on water, in swamps, on slopes, or within buildings. Difficult to obtain undisturbed samples.
Rotary drilling	Power rotation of drilling bit as circulating fluid removes cutting from hole. Changes indicated by rate of progress, action of drilling tools, and examination of cutting in drilling fluid. Casing usually not required except near surface.	Applicable to all soils except those containing much large gravel, cobbles, and boulders. Difficult to determine changes accurately in some soils. Not practical in inaccessible locations because of heavy truck mounted equipment, but applications are increasing since it is usually most

TABLE 5 (continued)
Types of Test Borings

Boring Method	Procedure Utilized	Applicability
		rapid method of advancing borehole. Soil samples and rock cores usually limited to 6 inches.
Percussion drilling (Churn drilling)	Power chopping with limited amount of water at bottom of hole. Water becomes a slurry that is periodically removed with bailer or sand pump. Changes indicated by rate of progress, action of drilling tools, and composition of slurry removed. Casing required except in stable rock.	Not preferred for ordinary exploration or where undisturbed samples are required because of difficulty in determining strata changes, disturbance caused below chopping bit, difficulty of access, and usually higher cost. Sometimes used in combination with auger or wash borings for penetration of coarse gravel, boulders, and rock formations. Could be useful to probe cavities and weakness in rock by changes in drill rate.
Rock core drilling	Power rotation of a core barrel as circulating water removes ground-up material from hole. Water also acts as coolant for core barrel bit. Generally hole is cased to rock.	Used alone and in combination with boring types to drill weathered rocks, bedrock, and boulder formations.
Wire-line drilling	Rotary type drilling method where the coring device is an integral part of the drill rod string which also serves as a casing. Core samples obtained by removing inner barrel assembly from the core barrel portion of the drill rod. The inner barrel is released by a retriever lowered by a wire-line through drilling rod.	Efficient for deep hole coring over 100 feet on land and offshore coring and sampling.

TABLE 6
Requirements for Boring Layout

Areas for Investigation	Boring Layout
New site of wide extent.	Space preliminary borings 200 to 500 ft apart so that area between any four borings includes approximately 10% of total area. In detailed exploration, add borings to establish geological sections at the most useful orientations.
Development of site on soft compressible strata.	Space borings 100 to 200 ft at possible building locations. Add intermediate borings when building sites are determined.
Large structure with separate closely spaced footings.	Space borings approximately 50 ft in both directions, including borings at possible exterior foundation walls at machinery or elevator pits, and to establish geologic sections at the most useful orientations.
Low-load ware-house building of large defined area.	Minimum of four borings at corners plus intermediate borings at interior foundations sufficient to subsoil profile.
Isolated rigid foundation 2,500 to 10,000 sq ft in area.	Minimum of three borings around perimeter. Add interior borings depending on initial results.
Isolated rigid foundation, less than 2,500 sq ft in area.	Minimum of two borings at opposite corners. Add more for erratic conditions.
Major waterfront structures, such as dry docks.	If definite site is established, space borings generally not farther than 50 ft adding intermediate borings at critical locations, such as deep pump-well gate seat, tunnel, or culverts.
Long bulkhead or wharf wall.	Preliminary borings on line of wall at 200 ft. spacing. Add intermediate borings to decrease spacing to 50 ft. Place certain intermediate borings inboard and outboard of wall line to determine materials in scour zone at toe and in active wedge behind wall.
Slope stability, deep cuts, high embankments.	Provide three to five borings on line in the critical direction to provide geological section for analysis. Number of geological sections depends on extent of stability problem. For an active slide, place at least one boring up slope of sliding area.

TABLE 6 (continued)
Requirements for Boring Layout

Areas for Investigation	Boring Layout
Dams and water retention structures.	Space preliminary borings approximately 200 ft over foundation area. Decrease spacing on centerline to 100 ft by intermediate borings. Include borings at location of cutoff, critical spots in abutment, spillway and outlet works.

TABLE 7
Requirements for Boring Depths

Areas of Investigation	Boring Depth
Large structure with separate closely spaced footings.	Extend to depth where increase in vertical stress for combined foundations is less than 10% of effective overburden stress. Generally all borings should extend to no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Isolated rigid foundations.	Extend to depth where vertical stress decreases to 10% of bearing pressure. Generally all borings should extend no less than 30 ft below lowest part of foundation unless rock is encountered at shallower depth.
Long bulkhead or wharf wall.	Extend to depth below dredge line between 3/4 and 1-1/2 times unbalanced height of wall. Where stratification indicates possible deep stability problem, selected borings should reach top of hard stratum.
Slope stability.	Extend to an elevation below active or potential failure surface and into hard stratum, or to a depth for which failure is unlikely because of geometry of cross section.
Deep cuts.	Extend to depth between 3/4 and 1 times base width of narrow cuts. Where cut is above groundwater in stable materials, depth of 4 to 8 ft below base may suffice. Where base is below groundwater, determine extent of pervious strata below base.
High embankments.	Extend to depth between 1/2 and 1-1/4 times horizontal length of side slope in relatively homogeneous foundation. Where soft strata are encountered, borings should reach hard materials.
Dams and water retention structures.	Extend to depth of 1/2 base width of earth dams or 1 to 1-1/2 times height of small concrete dams in relatively homogeneous foundations. Borings may terminate after penetration of 10 to 20 ft in hard and impervious stratum if continuity of this stratum is known from reconnaissance.

(2) Fine-Grained Strata. Extend borings in potentially compressible fine-grained strata of great thickness to a depth where stress from superposed load is 50 small that corresponding consolidation will not significantly influence surface settlement.

(3) Compact Soils. Where stiff or compact soils are encountered at shallow depths, extend boring(s) through this material to a depth where the presence of an underlying weaker stratum cannot affect stability or settlement.

(4) Bedrock Surface. If bedrock surface is encountered and general character and location of rock are known, extend one or two borings 5 feet into sound, unweathered rock. Where location and character of rock are unknown, or where boulders or irregularly weathered material are likely geologically, increase the number of borings penetrating into rock to bracket the area. In cavernous limestone areas, extend borings through strata suspected of containing solution channels.

(5) Check Borings. In unfamiliar areas, at least one boring should extend well below the zone necessary for apparent stability, to make sure no unusual conditions exist at greater depth.

d. Sealing Boreholes. Borings made in foundation areas that eventually will be excavated below groundwater, or where artesian pressures are encountered, must be plugged or grouted unless they are used for continuing water-level observations. In boreholes for groundwater observations, place casing in tight contact with walls of holes, or fill annular space with sand/gravel.

e. Cavernous Limestone. In limestone areas suspected of containing solution channels or cavities, each column location should be investigated. For smaller structures, locate boring or probe at each planned column location. For large structures and area investigation use indirect methods noted below, followed by borings or probes in final column locations, and on close centers (25 ft. under walls or heavily loaded areas). Aerial photographs have been used effectively by experienced geologists for detecting sinkholes and the progress of cavity development by comparing old to new photographs. Geophysical methods are used to detect anomalies in subsurface resistivity, gravity, magnetic field or seismic velocities and to correlate such anomalies with cavity presence (see Reference 5, The Use of Geophysical Methods in Engineering Geology, Part II, Electrical Resistivity, Magnetic and Gravity Methods, by Higginbottom, and Reference 6, Bedrock Verification Program for Davis Besse Nuclear Power Station, by Millet and Morehouse).

2. TEST PITS. Test pits are used to examine and sample soils in situ, to determine the depth to groundwater, and to determine the thickness of topsoil. They range from shallow manual or machine excavations to deep, sheeted, and braced pits. See Table 8 for types, uses, and limitations of test pits and trenches. Hand-cut samples are frequently necessary for highly sensitive, cohesive soils, brittle and weathered rock, and soil formation with honeycomb structure.

TABLE 8

Use, Capabilities and Limitations of Test Pits and Trenches

Exploration Method	General Use	Capabilities	Limitations
Hand-Excavated Test Pits and Shafts	Bulk sampling, in situ testing, visual inspection.	Provides data in inaccessible areas, less mechanical disturbance of surrounding ground.	Expensive, time-consuming, limited to depths above groundwater level.
Backhoe Excavated Test Pits and Trenches	Bulk sampling, in situ testing, visual inspection, excavation rates, depth of bedrock and groundwater.	Fast, economical, generally less than 15 feet deep, can be up to 30 feet deep.	Equipment access, generally limited to depths above groundwater level, limited undisturbed sampling.
Drilled Shafts	Pre-excavation for piles and shafts, landslide investigations, drainage wells.	Fast, more economical than hand excavated, min. 30 inches dia., max. 6 feet dia.	Equipment access, difficult to obtain undisturbed samples, casing obscures visual inspection.
Dozer Cuts	Bedrock characteristics depth of bedrock and groundwater level, rippability, increase depth capability of backhoes, level area for other exploration equipment.	Relatively low cost, exposures for geologic mapping.	Exploration limited to depth above groundwater level.
Trenches for Fault Investigations	Evaluation of presence and activity of faulting and sometimes landslide features.	Definitive location of faulting, subsurface observation up to 30 feet.	Costly, time-consuming, requires shoring, only useful where dateable materials are present, depth limited to zone above groundwater level.

3. TEST TRENCHES. Test trenches are particularly useful for exploration in very heterogeneous deposits such as rubble fills, where borings are either meaningless or not feasible. They are also useful for detection of fault traces in seismicity investigations.

Section 6. SAMPLING

1. APPLICATION. Disturbed samples are primarily used for classification tests and must contain all of the constituents of the soil even though the structure is disturbed. Undisturbed samples are taken primarily for laboratory strength and compressibility tests and in those cases where the in-place properties of the soil must be studied. Many offshore samplers fall in a special category and are treated separately in this section.

2. GENERAL REQUIREMENTS FOR SAMPLING PROGRAM. The number and type of samples to be taken depend on the stratification and material encountered.

a. Representative Disturbed Samples. Take representative disturbed samples at vertical intervals of no less than 5 feet and at every change in strata. Table 9 lists common types of samples for recovery of representative disturbed soil samples. Recommended procedures for obtaining disturbed samples are contained in ASTM Standard D1586, Penetration Test and Split Barrel Sampling of Soils.

b. Undisturbed Samples. The number and spacing of undisturbed samples depend on the anticipated design problems and the necessary testing program.

Undisturbed samples should comply with the following criteria: they should contain no visible distortion of strata, or opening or softening of materials; specific recovery ratio (length of undisturbed sample recovered divided by length of sampling push) should exceed 95 percent; and they should be taken with a sampler with an area ratio (annular cross-sectional area of sampling tube divided by full area of outside diameter of sampler) less than 15 percent. Table 10 lists common types of samplers used for recovery of representative undisturbed samples.

Obtain undisturbed samples in cohesive soil strata, so that there is at least one representative sample in each boring for each 10 feet depth. Recommended procedures for obtaining undisturbed samples are described in ASTM Standard D1587, Thin-Walled Tube Sampling of Soils. Additional cautions include the following:

(1) Caving. Use casing or viscous drilling fluid to advance borehole if there is danger of caving. If groundwater measurements are planned, drilling fluid should be of the revert type.

(2) Above Groundwater Table. When sampling above groundwater table, maintain borehole dry whenever possible.

TABLE 9

Common Samplers for Disturbed Soil Samples and Rock Cores

Sampler	Dimensions	Best Results in Soil or Rock Types	Methods of Penetration	Causes of Disturbance or Low Recovery	Remarks
Split Barrel	2" OD - 1.375" ID is standard. Penetrometer sizes up to 4" OD - 3.5" ID available.	All fine-grained soils in which sampler can be driven. Gravels invalidate drive data.	Hammer driven	Vibration	SPT is made using standard penetrometer with 140# hammer falling 30". Undisturbed samples often taken with liners. Some sample disturbance is likely.
Retractable Plug	1" OD tubes 6" long. Maximum of 6 tubes can be filled in single penetration.	For silts, clays, fine and loose sands.	Hammer driven	Improper soil types for sampler. Vibration.	Light weight, highly portable units can be hand carried to job. Sample disturbance is likely.
Augers:					
Continuous Helical Flight	3" to 16" dia. Can penetrate to depths in excess of 50 feet.	For most soils above water table. Will not penetrate hard soils or those containing cobbles or boulders.	Rotation	Hard soils, cobbles, boulders.	Rapid method of determining soil profile. Bag samples can be obtained. Log and sample depths must account for lag between penetration of bit and arrival of sample at surface.

TABLE 9 (continued)
Common Samplers for Disturbed Soil Samples and Rock Cores

Sampler	Dimensions	Best Results in Soil or Rock Types	Methods of Penetration	Causes of Disturbance or Low Recovery	Remarks
Disc	Up to 42" dia. Usually has maximum penetration of 25 feet.	Same as flight auger.	Rotation	Same as flight auger.	Rapid method of determining soil profile. Bag samples can be obtained.
Bucket	Up to 48" dia. common. Larger available. With extensions, depths greater than 80 feet are possible.	For most soils above water table. Can dig harder soil than above types, and can penetrate soils with cobbles and small boulders when equipped with a rock bucket.	Rotation	Soil too hard to dig.	Several type buckets available including those with ripper teeth and chopping buckets. Progress is slow when extensions are used.
Hollow Stem	Generally 6" to 8" OD with 3" to 4" ID hollow stem.	Same as Bucket.	Same	Same	A special type of flight auger with hollow center through which undisturbed samples or SPT can be taken.
Diamond Core Barrels	Standard sizes 1-1/2" to 3" OD, 7/8" to 2-1/8" core. See Figure 2. Barrel lengths 5 to 10 feet for exploration.	Hard rock. All barrels can be fitted with insert bits for coring soft rock or hard soil.			

TABLE 9 (continued)
Common Samplers for Disturbed Soil Samples and Rock Cores

Sampler	Dimensions	Best Results in Soil or Rock Types	Methods of Penetration	Causes of Disturbance or Low Recovery	Remarks
Single Tube		Primarily for strong, sound and uniform rock.		Fractured rock. Rock too soft.	Drill fluid must circulate around core - rock must not be subject to erosion. Single tube not often used for exploration.
Double Tube		Non-uniform, fractured, friable and soft rock.		Improper rotation or feed rate in fractured or soft rock.	Has inner barrel or swivel which does not rotate with outer tube. For soft, erodible rock. Best with bottom discharge bit.
Triple Tube		Same as Double Tube.		Same as Double Tube.	Differs from Double Tube by having an additional inner split tube liner. Intensely fractured rock core best preserved in this barrel.

TABLE 10

Common Samplers For Undisturbed Samples

Sampler	Dimensions	Best Results in soil types	Method of Penetration	Causes of Disturbance	Remarks
Shelby Tube	3" OD - 2.875" ID most common. Available from 2" to 5" OD. 30" sampler length is standard.	For cohesive fine-grained or soft soils. Gravelly soils will crimp the tube.	Pressing with fast, smooth stroke. Can be carefully hammered.	Erratic pressure applied during sampling, hammering, gravel particles, crimping tube edge, improper soil types for sampler.	Simplest sampler for undisturbed samples. Boring should be clean before lowering sampler. Little waste area in sampler. Not suitable for hard, dense or gravelly soils.
Stationary Piston	3" OD most common. Available from 2" to 5" OD. 30" sample length is standard.	For soft to medium clays and fine silts. Not for sandy soils.	Pressing with continuous, steady stroke.	Erratic pressure during sampling, allowing piston rod to move during press. Improper soil types for sampler.	Piston at end of sampler prevents entry of fluid and contaminating material. Requires heavy drill rig with hydraulic drill head. Generally less disturbed samples than Shelby. Not suitable for hard, dense or gravelly soil. No positive control of specific recovery ratio.

TABLE 10 (continued)
Common Samplers For Undisturbed Samples

Sampler	Dimensions	Best Results in soil types	Method of Penetration	Causes of Disturbance	Remarks
Hydraulic Piston (Osterberg)	3" OD most common - available from 2" to 4" OD, 36" sample length.	For silts-clays and some sandy soils.	Hydraulic or compressed air pressure.	Inadequate clamping of drill rods, erratic pressure.	Needs only standard drill rods. Requires adequate hydraulic or air capacity to activate sampler. Generally less disturbed samples than Shelby. Not suitable for hard, dense or gravelly soil. Not possible to limit length of push or amounts of sample penetration.
Denison	Samplers from 3.5" OD to 7-3/4" OD. (2.375" to 6.3" size samples). 24" sample length is standard.	Can be used for stiff to hard clay, silt and sands with some cementation, soft rock.	Rotation and hydraulic pressure.	Improperly operating sampler. Poor drilling procedures.	Inner tube face projects beyond outer tube which rotates. Amount of projection can be adjusted. Generally takes good samples. Not suitable for loose sands and soft clays.

TABLE 10 (continued)
Common Samplers For Undisturbed Samples

Sampler	Dimensions	Best Results in soil types	Method of Penetration	Causes of Disturbance	Remarks
Pitcher Sampler	Sampler 4.125" OD uses 3" Shelby Tubes. 24" sample length.	Same as Denison.	Same as Denison.	Same as Denison.	Differs from Denison in that inner tube projection is spring controlled. Often ineffective in cohesionless soils.
Hand cut block or cylindrical sample	Sample cut by hand.	Highest quality undisturbed sampling in cohesive soils, cohesionless soil, residual soil, weathered rock, soft rock.		Change of state of stress by excavation.	Requires accessible excavation. Requires dewatering if sampling below groundwater.

(3) Below Groundwater Table. When sampling below groundwater table, maintain borehole full of water or drilling fluid during cleanout, during sampling and sample withdrawal, and while removing cleanout tools. Where continuous samples are required, casing should remain full for the entire drilling and sampling operation.

(4) Soft or Loose Soil. Sampling of a soft or loose soil directly below a stiff or compact soil in the same tube should be avoided. Discontinue driving of sample tube when a sudden decrease in resistance occurs.

3. UNDISTURBED SAMPLES FROM TEST PITS. Hand trimmed samples may be obtained in test pits, in test trenches, or in surface exposures. Samples so obtained are potentially the least disturbed of all types of samples. The basic procedure consists of trimming out a column of soil the same size or slightly smaller than the container to be used in transportation, sliding the container over the sample, and surrounding the sample with wax. Tight, stiff containers that can be sealed, and are not readily distorted, should be used.

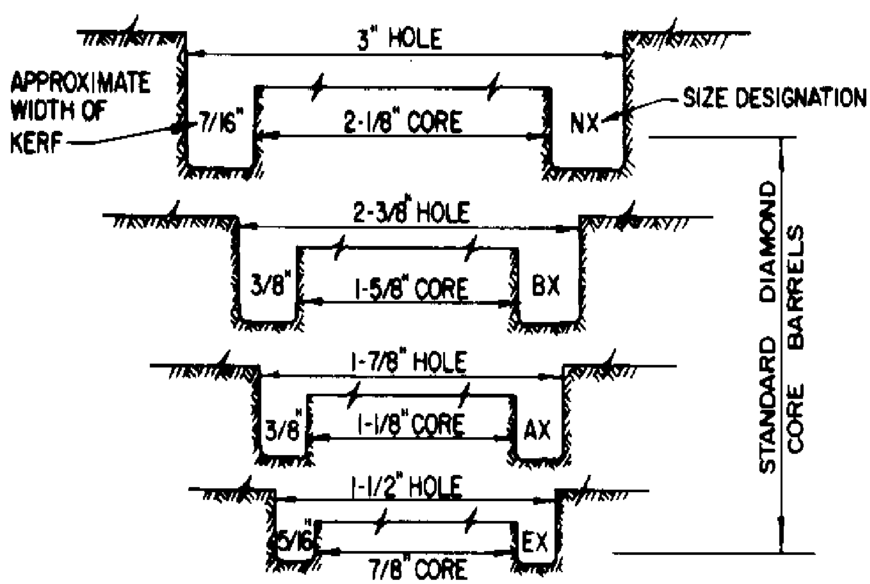
4. ROCK CORES. Rock is sampled with core barrels having either tungstencarbide or diamond core bits as listed or described in Table 9 and Figure 2.

The suitability of cores for structural property tests depends on the quality of individual samples. Specify double or triple tube core barrel for maximum core recovery in weathered, soft, or fractured rock. The percentage of core recovery is an indication of soundness and degree of weathering of rock. Carefully examine core section for reasons for low recovery. More details on rock recovery can be found in Chapter 1.

5. SAMPLING OF DISINTEGRATED ROCK TRANSITION ZONES. General guidance on sampling of rock with various degrees of disintegration is given in Table 11 (modified from Reference 7, Sampling of Residual Soils in Hong Kong, by Brenner).

6. OFFSHORE SAMPLING. For water depths less than about 60 feet, land type soil boring equipment can be used on small jack-up platforms, small barges or barrel floats. Floating equipment requires suitable anchoring and is limited to fairly calm sea conditions. For deeper water or more extreme seas, larger drill ships are required to obtain quality undisturbed samples. See Table 12 for common underwater samplers. Numerous types of oceanographic samplers, both open-tube and piston types, are available for use from shipboard. These depend upon free-fall penetration and thus are limited in depth of exploration. The quality of samples obtained by most oceanographic samplers is not high because of their large length to diameter ratio. For detailed information on underwater sampling equipment see Reference 8, Underwater Soil Sampling, Testing and Construction Control, ASTM STP 501, and Reference 9, Seafloor Soil Sampling and Geotechnical Parameter Determination - Handbook, by Lee and Clausner.

Size Symbol: Casing, Core Barrel	Size Symbol: Drill Rod	Casing OD	Casing Bit OD	Core Barrel Bit OD	Drill Rod OD	Approx. Diameter of Core Hole	Approx. Diameter of Core
EX	E	1 13/16	1 27/32	1 7/16	1 5/16	1 1/2	7/8
AX	A	2 1/4	2 5/16	1 27/32	1 5/8	1 7/8	1 1/8
BX	B	2 7/8	2/15/16	2 5/16	1 29/16	2 3/8	1 5/8
NX	N	3 1/2	3 9/16	2 15/16	2 3/8	3	2 1/8



STANDARDS BY NATIONAL BUREAU OF STANDARDS DIAMOND CORE DRILL MANUFACTURERS.

FIGURE 2
Standard Sizes, in Inches, for Casings, Rods, Core Barrels, and Holes

TABLE 11
Sampling of Disintegrated Rock Zones

Description of Material	Sampling Method
Colluvium - Loosely packed, poorly sorted material.	Driven samples or triple tube core barrel. Double tube barrel is required for boulders. Denison Sampler can be used if no boulders are present.
Structureless residual soil - The soil shows none of the fabric of the rock from which it is derived.	Driven samples or triple tube core barrel. Dennison Sampler can be used. Hand cut samples are best.
Decomposed rock containing rounded boulders which may be much harder than surrounding material.	Driven samples or triple tube core barrel. Double tube barrel is required to sample boulders.
Decomposed rock containing angular boulders separated by thin seams of friable material.	Double tube core barrel with triple tube barrel in weak seams.
Slightly decomposed rock - Friable material, if present, is limited to narrow seams.	Double tube core barrel.

TABLE 12

Common Underwater Samplers

Sampler	Size of Sample	Length of Sample	Water Depth Limitations	Method of Penetration	Remarks
Petersen Dredge	Grab	$\pm$ 6" depth.	To 200' and more with additional weight.	Clam shell jaw.	Reliable grab sampler, intact samples may be obtained with jaws that pre-cisely mate.
Open Barrel Gravity Corer	2.5" to 6" diameter	Core barrels length from 6' to 30'.	No limit on depth but required weight, amount of line or size of vessel may control.	Spooled freely off the winch drum.	
Phleger Corer	About 1.5" diameter	Core barrels available in 12", 24" and 36" length.	From 25' to 200'.	Free fall from 10' to 20' above bottom.	Relatively light-weight core for upper 1' to 3' of bottom sediments. Samples usually not suitable for strength tests.
Piston Gravity Corer	Standard corer has 2.5" barrel	Standard barrel is 10'. Addi-tional 10' sec-tions can be added.	No depth limit except that available weight, amount of line, or size of vessel may control.	Free fall from calibrated height above bottom such that piston does not penetrate sedi-ments.	Capable of obtaining samples suitable for strength tests with experienced crew, but samples may be seriously disturbed.

TABLE 12 (continued)
Common Underwater Samplers

Sampler	Size of Sample	Length of Sample	Water Depth Limitations	Method of Penetration	Remarks
Vibratory Corer	Sample is 3.5" diameter	20' standard, can be lengthened to 40'.	Minimum depth limited by draft of support vessel. Maximum depth about 200'.	Pneumatic impacting vibratory hammer.	Samples are disturbed because of vibration and large area ratio. Samples not suitable for strength testing. Penetration resistance can be measured, continuous representative samples in marine soils are obtained.

Section 7. PENETRATION RESISTANCE TESTS

1. GENERAL. The most common test is the Standard Penetration Test (SPT) which measures resistance to the penetration of a standard sampler in borings. The method is rapid, and when tests are properly conducted in the field, they yield useful data, although there are many factors which can affect the results. A more controlled test is the cone penetrometer test in which a cone shaped tip is jacked from the surface of the ground to provide a continuous resistance record.

a. Standard Penetration Test (SPT).

(1) Definition. The number of blows required to drive a split spoon sampler a distance of 12 inches after an initial penetration of 6 inches is referred to as an "N" value or SPT "N" value.

(2) Procedure. The test is covered under ASTM Standard D1586 which requires the use of a standard 2-inch (O.D.) split barrel sampler, driven by a 140 pound hammer dropping 30 inches in free fall. The procedure is generalized as follows:

(a) Clean the boring of all loose material, and material disturbed by drilling.

(b) Insert sampler, verifying the sampler reaches the same depth as was drilled.

(c) Obtain a consistent 30-inch free-fall drop of the hammer with two wraps of a rope around the cathead on the drill rig. (Cables attached to the hoisting drum should not be used because it is difficult to obtain free fall.)

(d) Drive the sampler 18 inches, or until normal maximum resistance (refusal) is reached, using the standard hammer and drop. (Refusal is defined as a penetration of less than 6 inches for 100 hammer blows.)

(e) Count and record the number of blows required to drive each 6 inches of penetration.

(3) Correlations. See Figure 1 and Table 4, Chapter 1 for approximate correlations between the "N" values from the standard penetration test and the compactness of granular soils and the consistency of fine grained soils.

(a) Relative Density of Granular (but fine grained) Deposits. Assuming that the test is a true standard test, the "N" value is influenced by the effective vertical stress at the level where "N" is measured, density of the soil, stress history, gradation and other factors. The work reported in Reference 10, SPT and Relative Density in Coarse Sands, by Marcuson and Bieganouski, establishes statistical relationships between relative density (D_r) in percent, "N" (blows/ft), effective vertical stress (pounds per square inch), gradation expressed in terms of uniformity coefficient (C_u), and overconsolidation ratio (OCR). The Gibbs & Holtz correlation of Figure 3 reported in Reference 11, Direct Determination and Indirect Evaluation of Relative Density and Earthwork Construction Projects, by Lacroix and Horn is commonly used to estimate the relative density from SPT.

(b) Undrained Shear Strength. A crude estimate for the undrained shear strength can be made using Figure 4. Correlations are not meaningful for medium to soft clays where effects of disturbance are excessive.

(c) Shear Modulus at Very Small Strains. A crude estimate of the shear modulus at small strains for sandy and cohesive soils can be obtained from the statistical relationships in Figure 5 (Reference 12, On Dynamic Shear Moduli and Poisson's Ratios of Soil Deposits, by Ohsaki and Iwasaki).

(d) Limitations. Except where confirmed by specific structural property tests, these relationships are suitable for estimates only. Blow counts are affected by operational procedures, by the presence of gravel, or cementation. They do not reflect fractures or slickensides in clay, which may be very important to strength characteristics. The standard penetration test results (N values) are influenced by operational procedures as illustrated in Table 13 (modified from Reference 13, Properties of Soil and Rock, by the Canadian Geotechnical Society).

b. Cone Penetrometer Tests (CPT). This test involves forcing a cone into the ground and measuring the rate of pressure needed for each increment of penetration. (See Figure 6). The most commonly used cone test is the Dutch Cone Test (DCT).

(1) Resistance. For the Dutch Cone, resistance to penetration is the sum of point resistance and frictional resistance on the sides of the shaft. The more sophisticated systems can differentiate between the point and frictional components of the resistance, and the ratio between frictional and point resistance (Friction Ratio) is one aid in differentiating between various soil types. Clean sands generally exhibit very low ratios (low friction component in comparison to point resistance), while an increase in clay content will usually result in a higher ratio, more often the result of a reduction in point resistance rather than an increase in frictional component.

(2) Correlations. Correlations have been developed for the cone penetration test with bearing capacity, relative density of sands, strength and sensitivity of clays and overconsolidation, as well as with SPT values and pile design parameters. Procedures and limitations of the cone penetration test and its correlations are described in Reference 14, Guidelines for Cone Penetration Tests Performance and Design, Federal Highway Administration.

(3) Advantages and Limitations. The static cone test can be used as a partial replacement for conventional borings. The speed of operation allows considerable data to be obtained in a short period of time. The major drawbacks of static cone tests are the non-recoverability of samples for identification, difficulty in advancing the cone in dense or hard deposits, and need for stable and fairly strong working surface to jack the rig against.

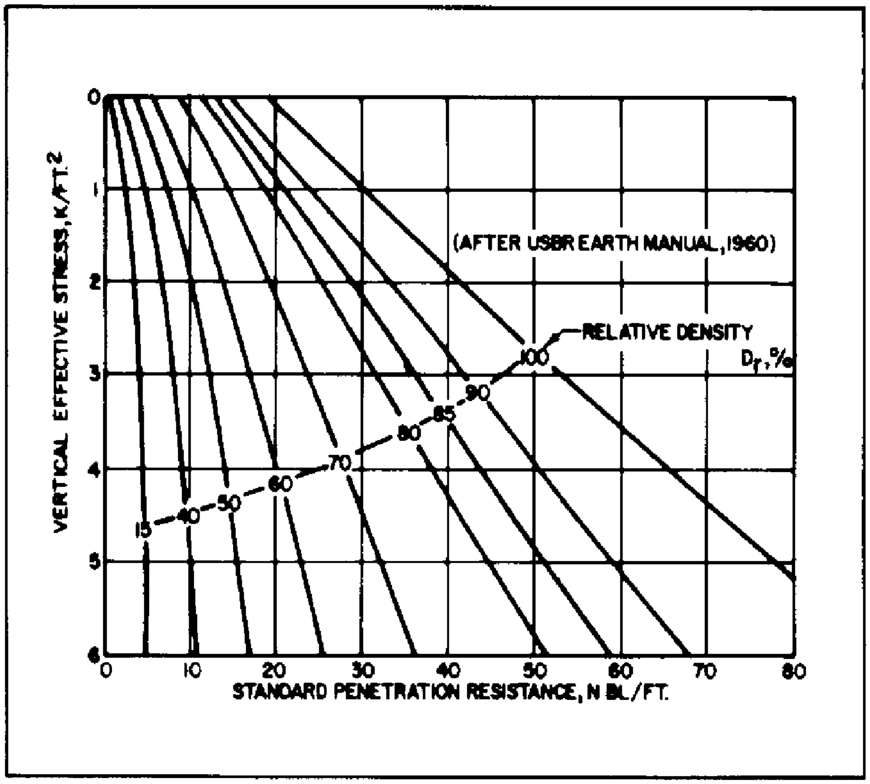


FIGURE 3
Correlations Between Relative Density and Standard Penetration Resistance in Accordance with Gibbs and Holtz

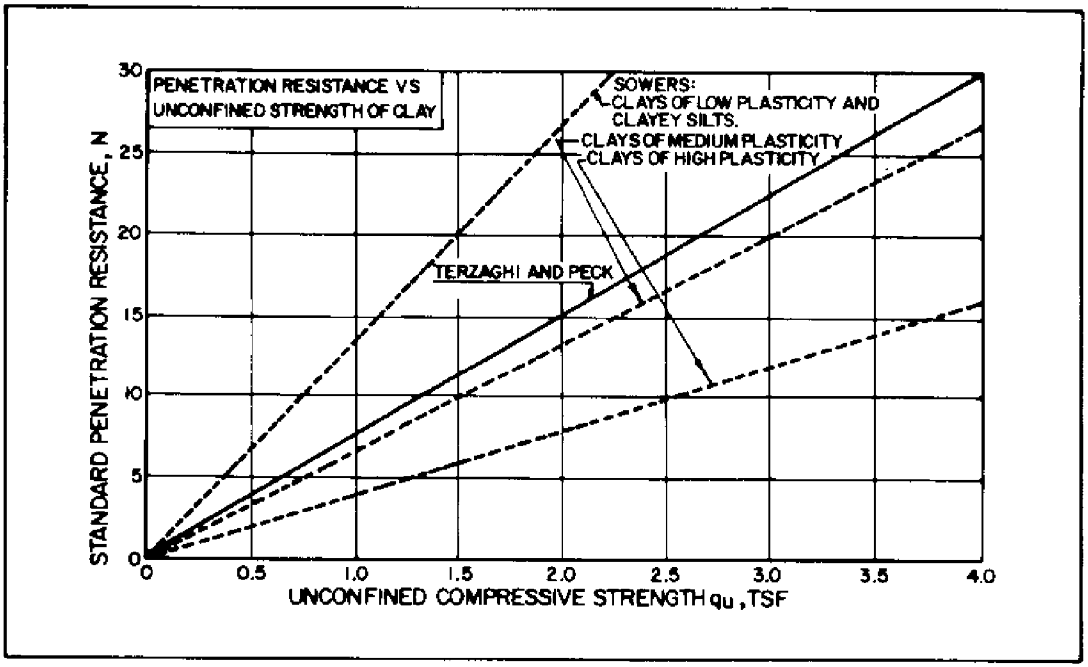


FIGURE 4
Correlations of Standard Penetration Resistance

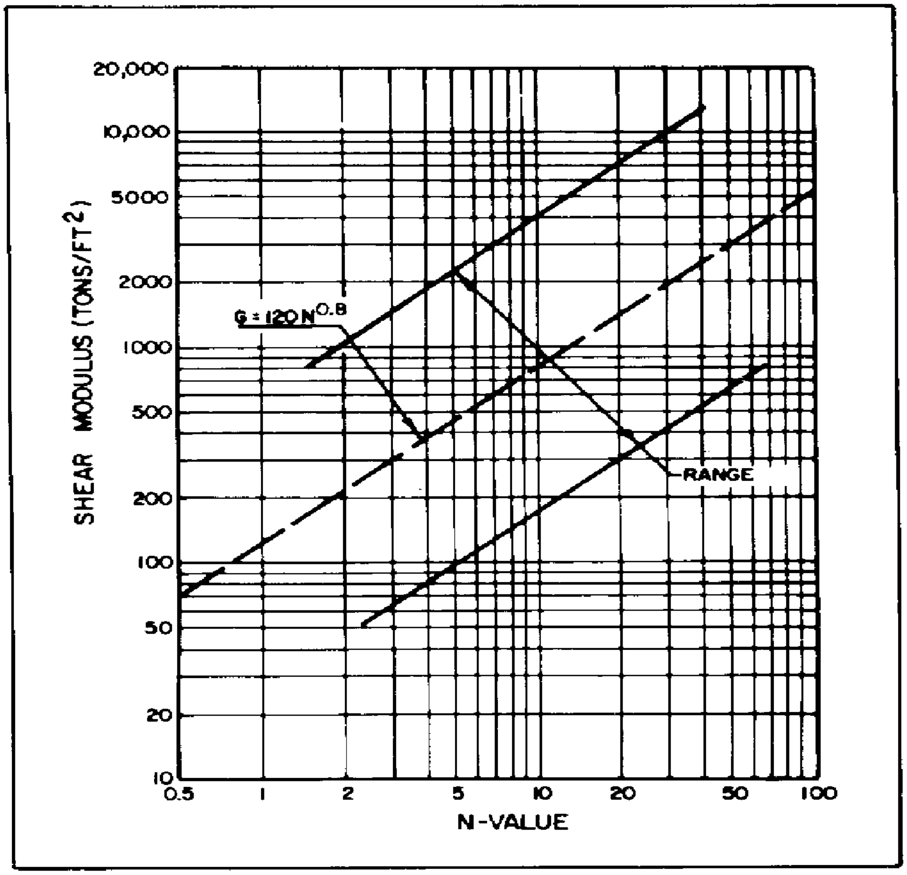


FIGURE 5
Shear Modulus vs. N Values (SPT) at Very Small Strains

TABLE 13
Procedures Which May Affect the Measured "N" Values

Inadequate cleaning of the borehole	SPT is only partially made in original soil. Sludge may be trapped in the sampler and compressed as the sampler is driven, increasing the blow count. (This may also prevent sample recovery.)
Not seating the sampler spoon on undisturbed material	Incorrect "N" values obtained.
Driving of sthe sample spoon above the bottom of the casing	"N" values are increased in sands and reduced in cohesive soils.
Failure to maintain sufficient hydrostatic head in boring	The water table in the borehole must be at least equal to the piezometric level in the sand, otherwise the sand at the bottom of the borehole may be transformed into a loose state.
Attitude of operators	Blow counts for the same soil using the same rig can vary, depending on who is operating the rig, and perhaps the mood of operator and time of drilling.
Overdrive sampler sampler	Higher blow counts usually result from overdriven sampler.
Sampler plugged by gravel	Higher blow counts result when gravel plugs sampler, resistance of loose sand could be highly overestimated.
Plugged casing	High "N" values may be recorded for loose sand when sampling below groundwater table. Hydrostatic pressure causes sand to rise and plug casing.
Overwashing ahead of casing	Low blow count may result for dense sand since sand is loosened by overwashing.
Drilling method	Drilling technique (e.g., cased holes vs. mud stabilized holes) may result in different "N" values for the same soil.
Not using the standard hammer drop	Energy delivered per blow is not uniform. European countries have adopted an automatic trip hammer not currently in use in North America.
Free fall of the drive weight is not attained	Using more than 1-1/2 turns of rope around the drum and/or using wire cable will restrict the fall of the drive weight.

TABLE 13 (continued)
Procedures Which May Affect the Measured "N" Values

Not using correct weight	Driller frequently supplies drive hammers with weights varying from the standard by as much as 10 lbs.
Weight does not strike the drive cap concentrically	Impact energy is reduced, increasing "N" values.
Not using a guide rod	Incorrect "N" value obtained.
Not using a good tip on the sampling spoon	If the tip is damaged and reduces the opening or increases the end area the" N" value can be increased.
Use of drill rods heavier than standard	With heavier rods more energy is absorbed by the rods causing an increase in the blow count.
Not recording blow counts and penetration accurately	Incorrect "N" values obtained.
Incorrect drilling procedures	The SPT was originally developed from wash boring techniques. Drilling procedures which seriously disturb the soil will affect the "N" value, e.g. drilling with cable tool equipment.
Using drill holes that are too large	Holes greater than 10 cm (4 in) in diameter are not recommended. Use of larger diameters may result in decreases in the blow count.
Inadequate supervision	Frequently a sampler will be impeded by gravel or cobbles causing a sudden increase in blow count; this is not recognized by an inexperienced observer. (Accurate recording of drilling, sampling and depth is always required.)
Improper logging of soils	Not describing the sample correctly.
Using too large a pump	Too high a pump capacity will loosen the soil at the base of the hole causing a decrease in blow count.

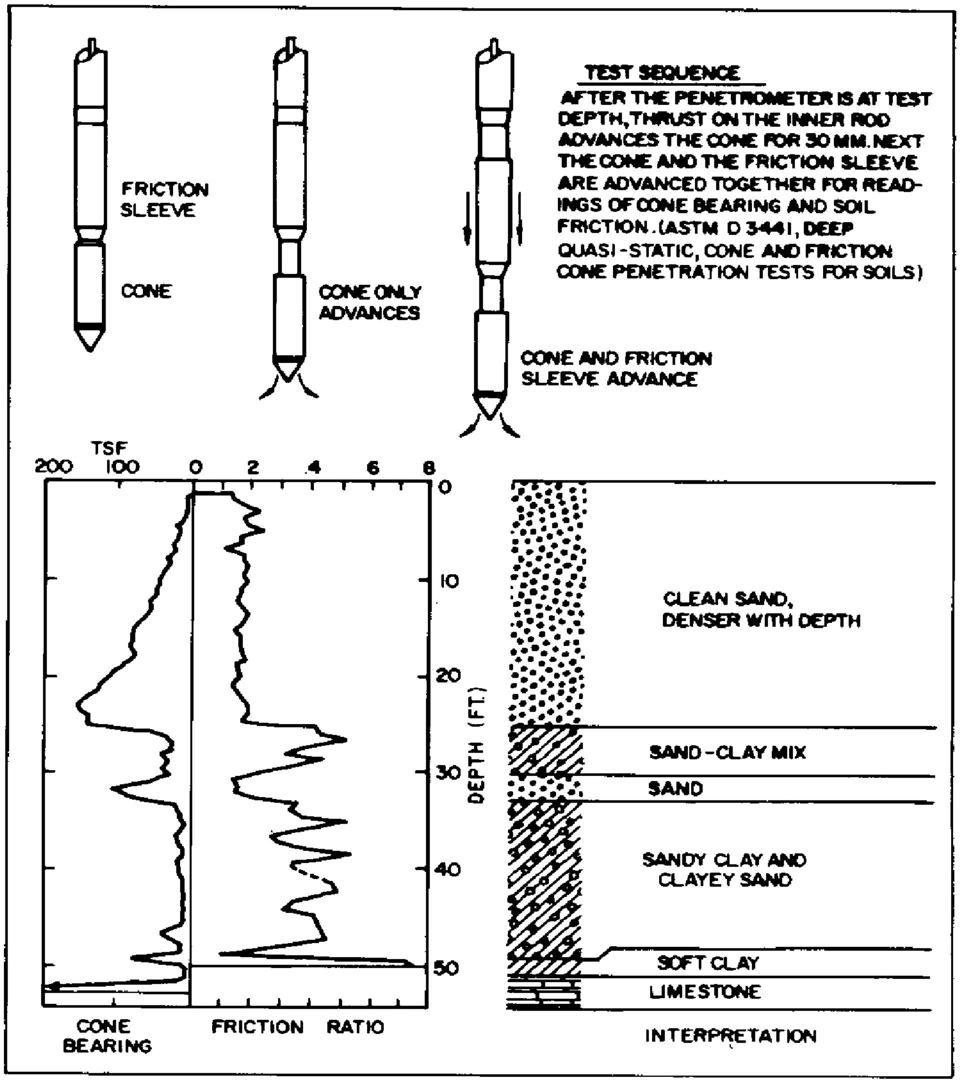


FIGURE 6
Dutch Cone Penetrometer

Section 8. GROUNDWATER MEASUREMENTS

1. UTILIZATION. The groundwater level should be measured at the depth at which water is first encountered as well as at the level at which it stabilizes after drilling. If necessary, the boring should be kept open with perforated casing until stabilization occurs. On many projects, seasonal groundwater fluctuation is of importance and long-term measurements can be made by converting the borings to standpipe piezometers. For certain construction projects, more sophisticated pneumatic or electrical types of piezometers may be used.

2. TYPICAL INSTALLATION. The three basic components of a piezometer installation are:

a. Tip. A piezometer tip consisting of a perforated section, well screen, porous tube, or other similar feature and, in fine-grained or unstable materials, a surrounding zone of filter sand;

b. Standpipe. Watertight standpipe or measurement conduit, of the smallest practical diameter, attached to the tip and extending to the surface of the ground;

c. Seals. A seal or seals consisting of cement grout, bentonite slurry, or other similarly impermeable material placed between the standpipe and the boring walls to isolate the zone to be monitored.

The vertical location, i.e., depth and elevation of each item must be accurately measured and recorded.

3. PIEZOMETER TYPES. All systems, except the open well, have a porous filter element which is placed in the ground. The most common types used for groundwater measurements are described below (see Table 14).

a. Open Well. The most common groundwater recording technique is to measure water level in an open boring as shown in Figure 7(a). A disadvantage is that different layers of soil may be under different hydrostatic pressures and therefore the groundwater level recorded may be inaccurate and misleading. Thus, this system is useful only for relatively homogeneous deposits.

(1) Open Standpipe Piezometer. Most of the disadvantages of the open borehole can be overcome by installing an open standpipe piezometer in the borehole as shown in Figure 7(b). This system is effective in isolating substrata of interest.

b. Porous Element Piezometer. As shown in Figure 8, a porous element is connected to the riser pipe which is of small diameter to reduce the equalization time. The most common tip is the nonmetallic ceramic stone (Casagrande Type). The ceramic tip is subject to damage and for that reason porous metal tips or other tips of the same dimension are now available. Pores are about 50 microns size, so that the tip can be used in direct contact with fine-grained soils.

TABLE 14
Groundwater or Piezometric Level Monitoring Devices

Instrument	Advantages	Disadvantages
Standpipe piezometer or wellpoint.	Simple. Reliable. Long experience record. No elaborate terminal point needed.	Slow response time. Freezing problems.
Pneumatic piezometer.	Level of terminal independent of tip level. Rapid response.	Must prevent humid air from entering tubing.
Electric piezometer	Level of terminal independent of tip level. Rapid response. High sensitivity. Suitable for automatic readout.	Expensive. Temperature correction may be required. Errors due to zero drift can arise.

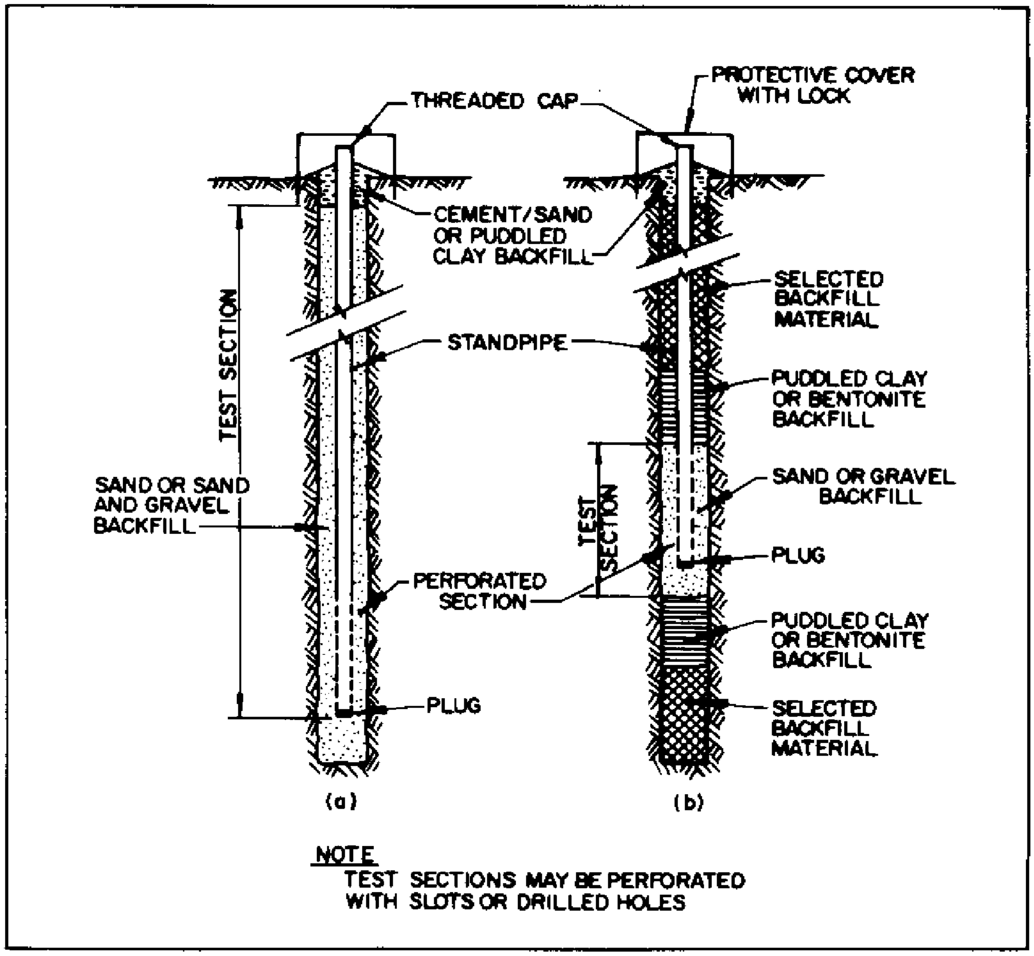


FIGURE 7
Open Standpipe Piezometers

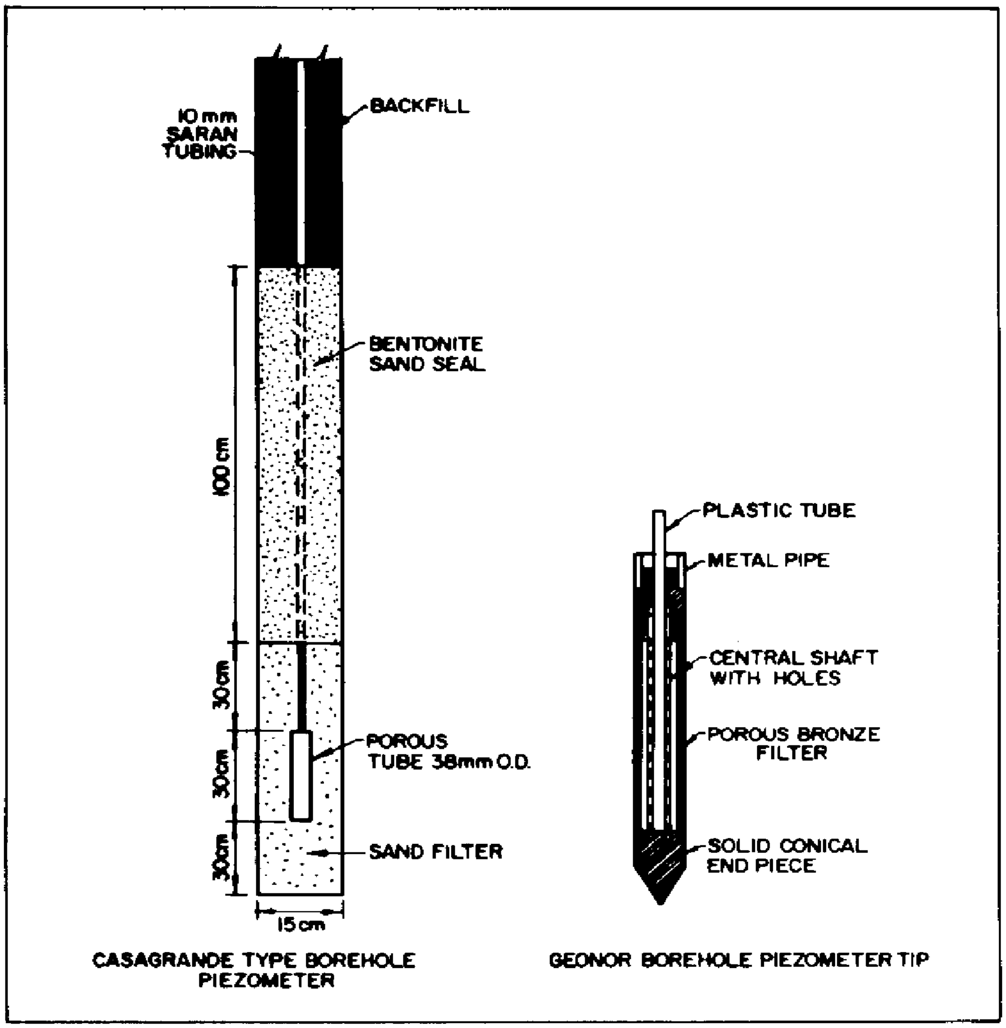


FIGURE 8
Porous Element Piezometers

c. Other Types. Other piezometers used for special investigations include electrical, air pneumatic, oil pneumatic and water pressure types.

4. MULTIPLE INSTALLATIONS. Several piezometers may be installed in a single boring with an impervious seal separating the measuring zones. However, if measurements are needed in zones with 10 feet or less of vertical separation, it is generally best to install piezometers in separate borings.

5. MEASUREMENTS. water levels can be measured to within 0.5 inch, using several devices, including the plumb bob, cloth or metal surveyors' tapes coated with chalk, or commercially available electrical indicators for use in small tubes.

6. SOURCES OF ERROR. Major sources of error are due to gas bubbles and tube blockage. Some are shown in Figure 9. The magnitude of errors can be controlled by proper piezometer selection, installation, and de-airing techniques.

Section 9. MEASUREMENT OF SOIL AND ROCK PROPERTIES IN SITU

1. SCOPE. A great number of tools and methods have been devised for measuring in situ engineering properties of soil and rock. The most common tools, the split spoon sampler and the cone penetrometer, have been previously discussed. This section describes other methods commonly used in exploration programs or during construction control.

2. SHEAR STRENGTH BY DIRECT METHODS. Several devices are available to obtain shear strength data in the field as a supplement to laboratory tests or where it is not possible to obtain representative samples for testing.

a. Pocket Penetrometer. Used for obtaining the shear strength of cohesive, non-gravelly soils on field exploration or construction sites. Commercial penetrometers are available which read unconfined compressive strength directly. The tool is used as an aid to obtaining uniform classification of soils. It does not replace other field tests or laboratory tests.

b. Torvane Shear Device. Used for obtaining rapid approximations of shear strength of cohesive, non-gravelly soils on field exploration. Can be used on ends of Shelby tubes, penetration samples, block samples from test pits or sides of test pits. The device is used in uniform soils and does not replace laboratory tests.

c. Vane Shear Apparatus. Equipment setup for the vane shear test is illustrated in Figure 10 (Reference 15, Acker Soil Sampling Catalog, by Acker Drill Company, Inc.). In situ vane shear measurements are especially useful in very soft soil deposits where much of the strength may be lost by disturbance during sampling. It should not be used in stiff clays or in soft soils containing gravel, shells, wood, etc. The main equipment components are the torque assembly, which includes a gear reduction device capable of producing constant angular rotation of 1 degree to 6 degrees per minute, a calibrated proving ring with a dial gage for torque measurement within 5%, a means of

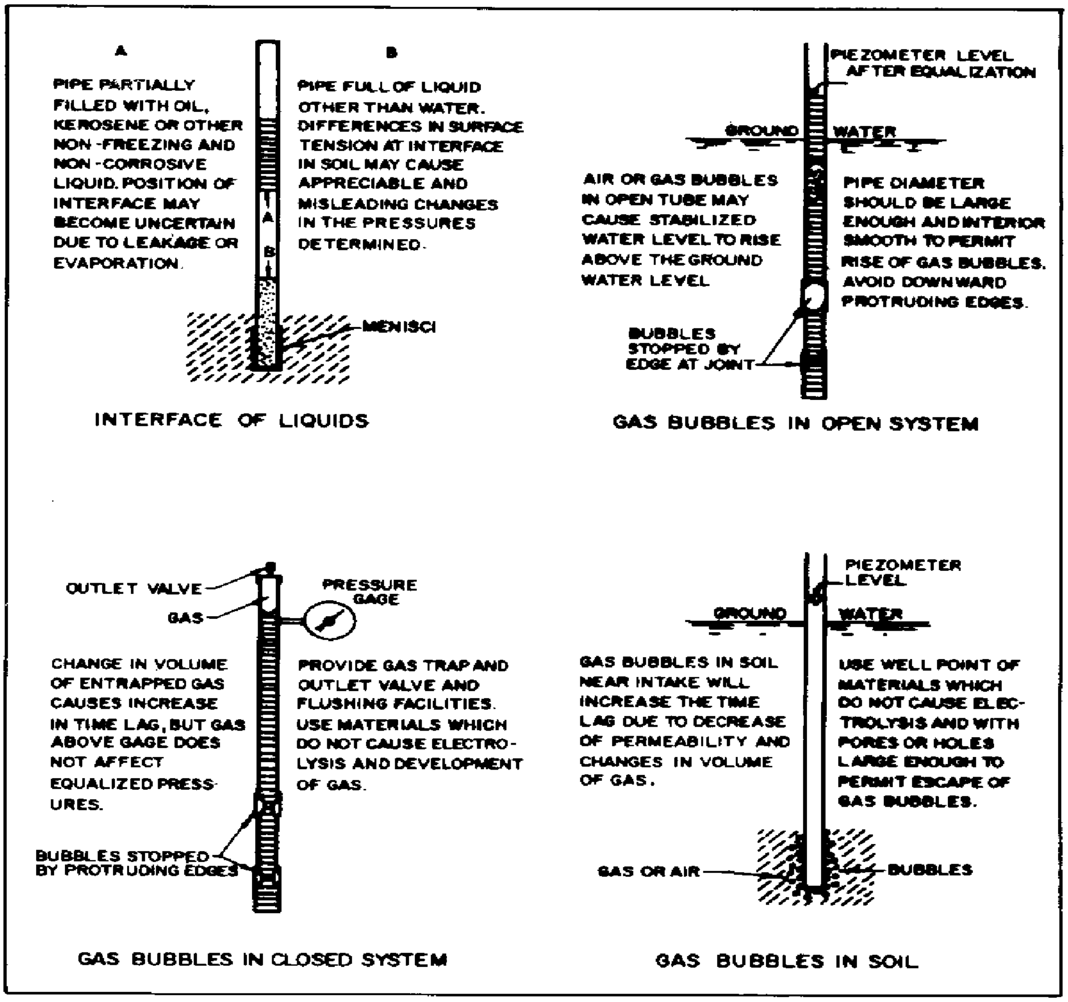


FIGURE 9
Sources of Error and Corrective Methods in
Groundwater Pressure Measurements

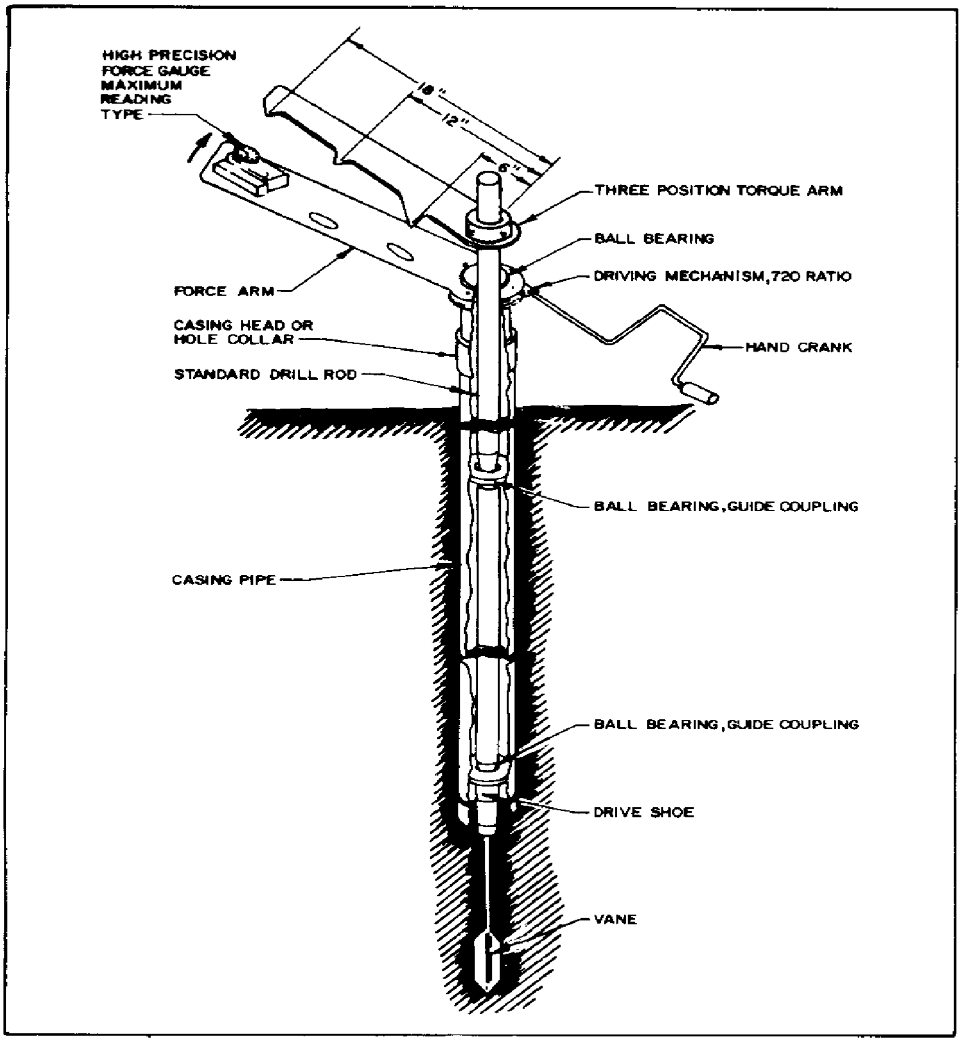


FIGURE 10
Vane Shear Test Arrangement

measuring angular rotation in degrees, and thrust bearings to support vane at ground surface. Procedures for the vane shear test and methods of interpretation are described under ASTM Standard D2573, Field Vane Shear Test in Cohesive Soil.

3. DEFORMATION MODULI. A number of different methods are available for obtaining values of deformation moduli in soil and rock. Each method has its own advantages or disadvantages and in situ testing should only be attempted with a full knowledge of the limitations of the several techniques.

a. Pressuremeter. See Figure 11 (modified from Reference 13). The pressuremeter test is an in situ lateral loading test performed in a borehole by means of a cylindrical probe. Under increments of pressure, radial expansion is measured, and the modulus of deformation is calculated. If the test is carried to failure, shear strengths can be calculated and are generally higher than those obtained from vane shear tests. Materials difficult to sample (e.g., sands, residual soil, tills, soft rock) can be effectively investigated by the pressuremeter. Equipment and procedures for the pressuremeter are described in Reference 13.

(1) Limitations. Pressuremeter tests are sensitive to test procedures. The tests measure soil compressibility in the radial direction and some assumptions are required on the ratio between the vertical moduli to radial moduli. This may be difficult to interpret and thus of only limited value for stratified soils, for very soft soils, and for soils where drainage conditions during loading are not known. Roughness of the borehole wall affects test results, although the self-boring pressuremeter eliminates some of this disadvantage (see Reference 16, French Self-Boring Pressuremeter, by Baguelin and Jezequal, and Reference 17, Cambridge In-Situ Probe, by Wroth).

b. Plate Bearing Test. The plate bearing test can be used as an indicator of compressibility and as a supplement to other compressibility data.

(1) Procedure. For ordinary tests for foundation studies, use procedure of ASTM Standard D1194, Test for Bearing Capacity Of Soil for Static Load on Spread Footings, except that dial gages reading to 0.001 in. should be substituted. Tests are utilized to estimate the modulus of subgrade reaction and settlements of spread foundations. Results obtained have no relation to deep seated settlement from volume change under load of entire foundation.

(2) Analysis of Test Results. (See Figure 12.) Determine yield point pressure for logarithmic plot of load versus settlement. Convert modulus of subgrade reaction determined from test K+vi, to the property K+v, for use in computing immediate settlement (Chapter 5). In general, tests should be conducted with groundwater saturation conditions simulating those anticipated under the actual structure.

Data from the plate load test is applicable to material only in the immediate zone (say to a depth of two plate diameters) of the plate and should not be extrapolated unless material at greater depth is essentially the same.

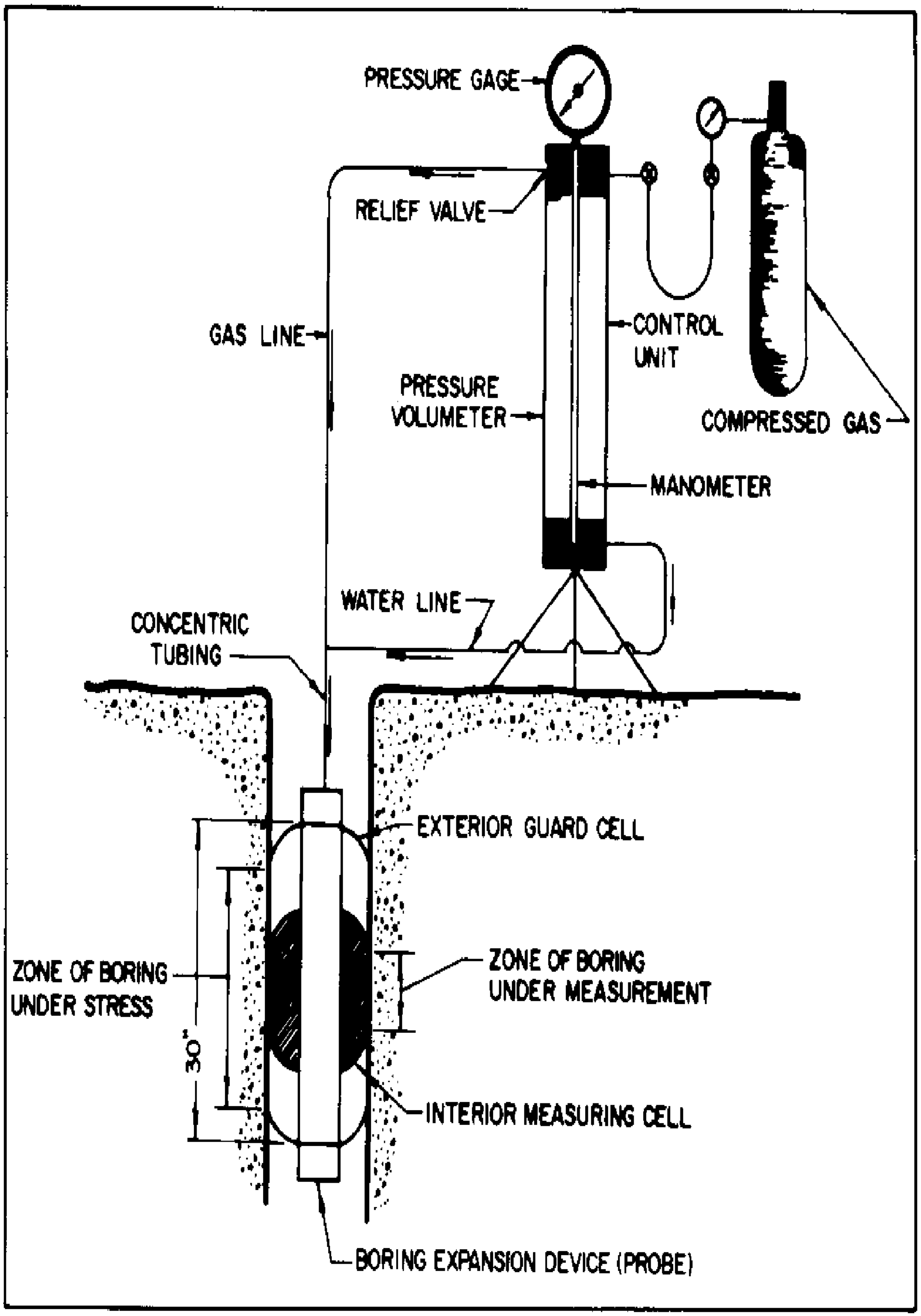


FIGURE 11
Menard Pressuremeter Equipment

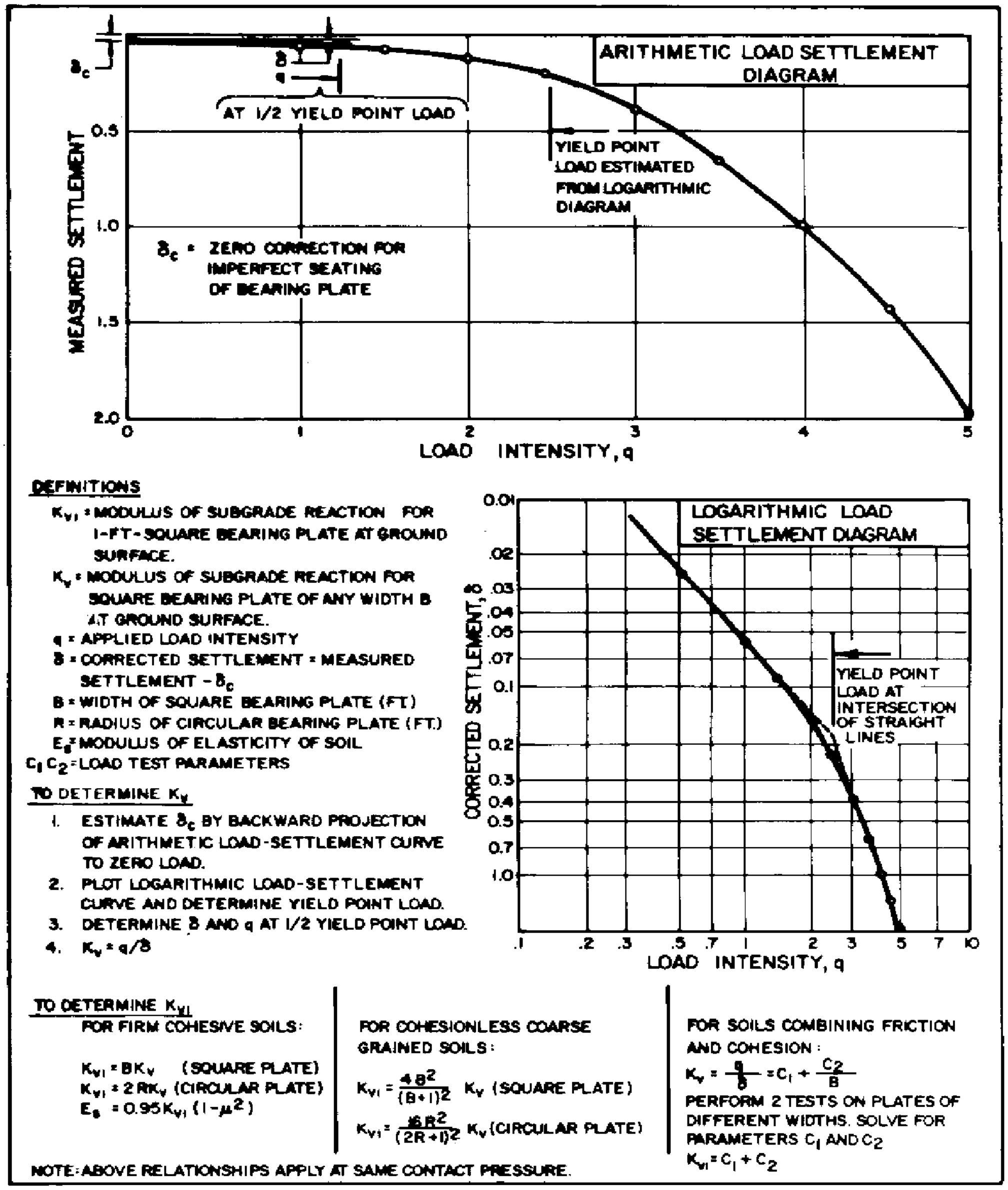


FIGURE 12
Analysis of Plate Bearing Tests

4. PERMEABILITY. Field permeability tests measure the coefficient of permeability (hydraulic conductivity) of in-place materials. The coefficient of permeability is the factor of proportionality relating the rate of fluid discharge per unit of cross-sectional area to the hydraulic gradient (the pressure or "head" inducing flow, divided by the length of the flow path). This relation is usually expressed simply:

$$Q/A = \frac{HK}{L}$$

Where Q is discharge (volume/time); A is cross-sectional area, H/L is the hydraulic gradient (dimensionless); and K is the coefficient of permeability· expressed in length per unit time (cm/sec, ft/day, etc.). The area and length factors are often combined in a "shape factor" or "conductivity coefficient." See Figure 13 for analysis of observations and Table 15 for methods of computation. Permeability is the most variable of all the material properties commonly used in geotechnical analysis. A permeability spread of ten or more orders of magnitude has been reported for a number of different types of tests and materials. Measurement of permeability is highly sensitive to both natural and test conditions. The difficulties inherent in field permeability testing require that great care be taken to minimize sources of error and to correctly interpret, and compensate for, deviations from ideal test conditions.

a. <u>Factors Affecting Tests</u>. The following five physical characteristics influence the performance and applicability of permeability tests:

(1) position of the water level,

(2) type of material - rock or soil,

(3) depth of the test zone,

(4) permeability of the test zone, and

(5) heterogeneity and anisotropy of the test zone.

To account for these it is necessary to isolate the test zone. Methods for doing so are shown in Figure 14.

b. <u>Types of Tests</u>. Many types of field permeability tests can be performed. In geotechnical exploration, equilibrium tests are the most common. These include constant and variable head gravity tests and pressure (Packer) tests conducted in single borings. In a few geotechnical investigations, and commonly in water resource or environmental studies, non-equilibrium "aquifer" or "pump" tests are conducted (a well is pumped at a constant rate for an extended period of time). See Table 15 for computation of permeability from variable head tests.

(1) Constant Head Test. This is the most generally applicable permeability test. It may be difficult to perform in materials of either very high or very low permeability since the flow of water may be difficult to maintain or to measure.

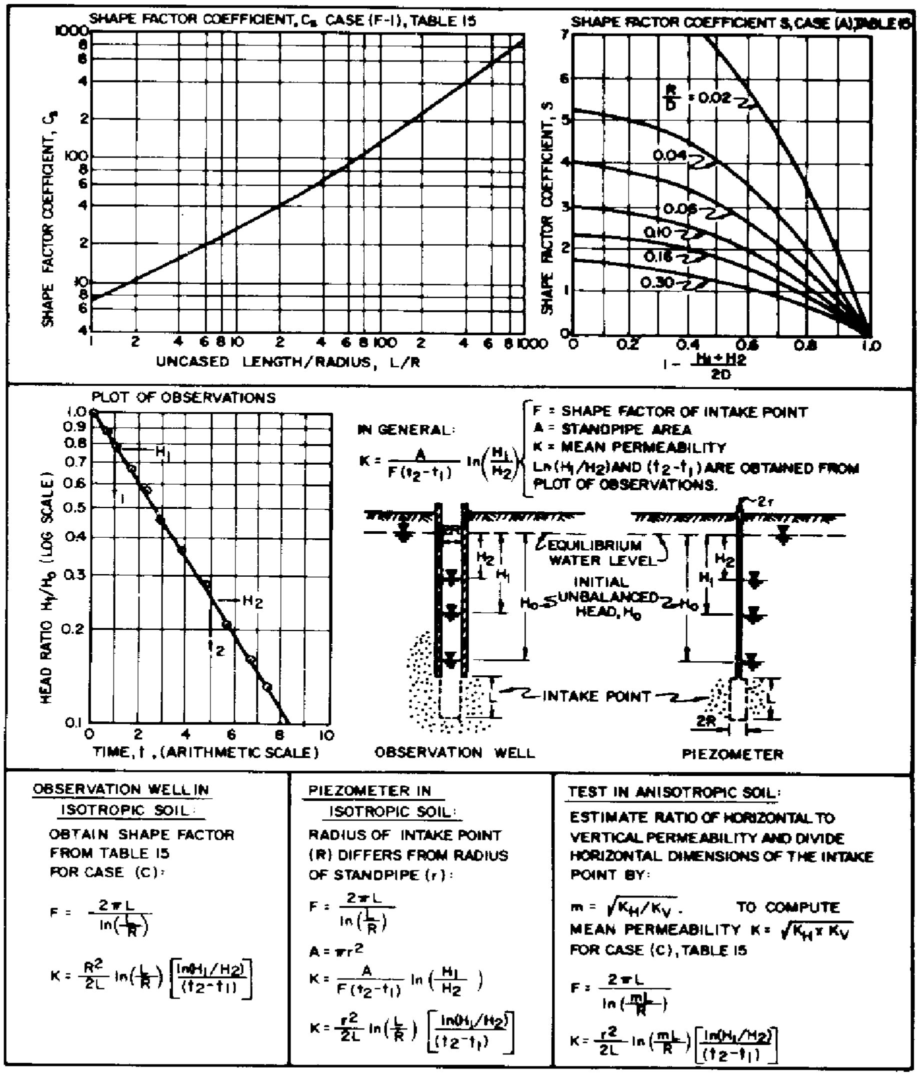


FIGURE 13
Analysis of Permeability by Variable Head Tests

TABLE 15
Shape Factors for Computation of Permeability From Variable Head Tests

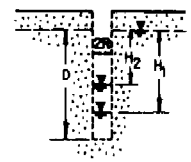
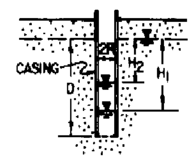
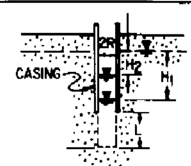
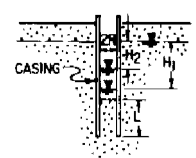
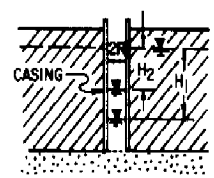
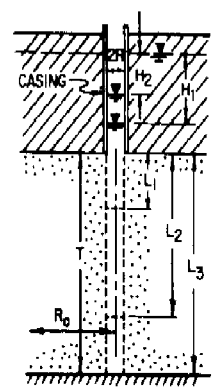
	CONDITION	DIAGRAM	SHAPE FACTOR, F	PERMEABILITY, K BY VARIABLE HEAD TEST	APPLICABILITY
OBSERVATION WELL OR PIEZOMETER IN SATURATED ISOTROPIC STRATUM OF INFINITE DEPTH	(A) UNCASED HOLE		$F = 16\pi DSR$	(FOR OBSERVATION WELL OF CONSTANT CROSS SECTION) $K = \frac{R}{16DS} \times \frac{(H_2 - H_1)}{(t_2 - t_1)}$ FOR $\frac{D}{R} < 50$	SIMPLEST METHOD FOR PERMEABILITY DETERMINATION. NOT APPLICABLE IN STRATIFIED SOILS. FOR VALUES OF S, SEE FIGURE 13.
	(B) CASED HOLE, SOIL FLUSH WITH BOTTOM.		$F = \frac{11R}{2}$	$K = \frac{2\pi R}{11(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$ FOR $6'' \leq D \leq 60''$	USED FOR PERMEABILITY DETERMINATION AT SHALLOW DEPTHS BELOW THE WATER TABLE. MAY YIELD UNRELIABLE RESULTS IN FALLING HEAD TEST WITH SILTING OF BOTTOM OF HOLE.
	(C) CASED HOLE, UNCASED OR PERFORATED EXTENSION OF LENGTH "L".		$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$	$K = \frac{R^2}{2L(t_2 - t_1)} \ln\left(\frac{L}{R}\right) \ln\left(\frac{H_1}{H_2}\right)$ FOR $\frac{L}{R} > 8$	USED FOR PERMEABILITY DETERMINATIONS AT GREATER DEPTHS BELOW WATER TABLE.
	(D) CASED HOLE, COLUMN OF SOIL INSIDE CASING TO HEIGHT "L"		$F = \frac{11\pi R^2}{2\pi R + 11L}$	$K = \frac{2\pi R + 11L}{11(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	PRINCIPAL USE IS FOR PERMEABILITY IN VERTICAL DIRECTION IN ANISOTROPIC SOILS.

TABLE 15 (continued)
Shape Factors for Computation of Permeability From Variable Head Tests

	CONDITION	DIAGRAM	SHAPE FACTOR, F	PERMEABILITY, K BY VARIABLE HEAD TEST	APPLICABILITY
OBSERVATION WELL OR PIEZOMETER IN AQUIFER WITH IMPERVIOUS UPPER LAYER	(E) CASED HOLE, OPENING FLUSH WITH UPPER BOUNDARY OF AQUIFER OF INFINITE DEPTH.		$F = 4R$	(FOR OBSERVATION WELL OF CONSTANT CROSS SECTION) $K = \frac{\pi R}{4(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	USED FOR PERMEABILITY DETERMINATION WHEN SURFACE IMPERVIOUS LAYER IS RELATIVELY THIN. MAY YIELD UNRELIABLE RESULTS IN FALLING HEAD TEST WITH SILTING OF BOTTOM OF HOLE.
	(F) CASED HOLE, UNCASED OR PERFORATED EXTENSION INTO AQUIFER OF FINITE THICKNESS: (1) $\frac{L_1}{T} \leq 0.2$ (2) $0.2 < \frac{L_2}{T} < 0.85$ (3) $\frac{L_3}{T} = 1.00$ NOTE: R_0 EQUALS EFFECTIVE RADIUS TO SOURCE AT CONSTANT HEAD.		(1) $F = C_s R$	$K = \frac{\pi R}{C_s(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	USED FOR PERMEABILITY DETERMINATIONS AT DEPTHS GREATER THAN ABOUT 5FT. FOR VALUES OF C_s, SEE FIGURE 13.
			(2) $F = \frac{2\pi L_2}{\ln(L_2/R)}$	$K = \frac{R^2 \ln\left(\frac{L_2}{R}\right)}{2L_2(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$ FOR $\frac{L}{R} > 8$	USED FOR PERMEABILITY DETERMINATIONS AT GREATER DEPTHS AND FOR FINE GRAINED SOILS USING POROUS INTAKE POINT OF PIEZOMETER.
			(3) $F = \frac{2\pi L_3}{\ln\left(\frac{R_0}{R}\right)}$	$K = \frac{R^2 \ln\left(\frac{R_0}{R}\right)}{2L_3(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	ASSUME VALUE OF $\frac{R_0}{R}$ = 200 FOR ESTIMATES UNLESS OBSERVATION WELLS ARE MADE TO DETERMINE ACTUAL VALUE OF R_0.

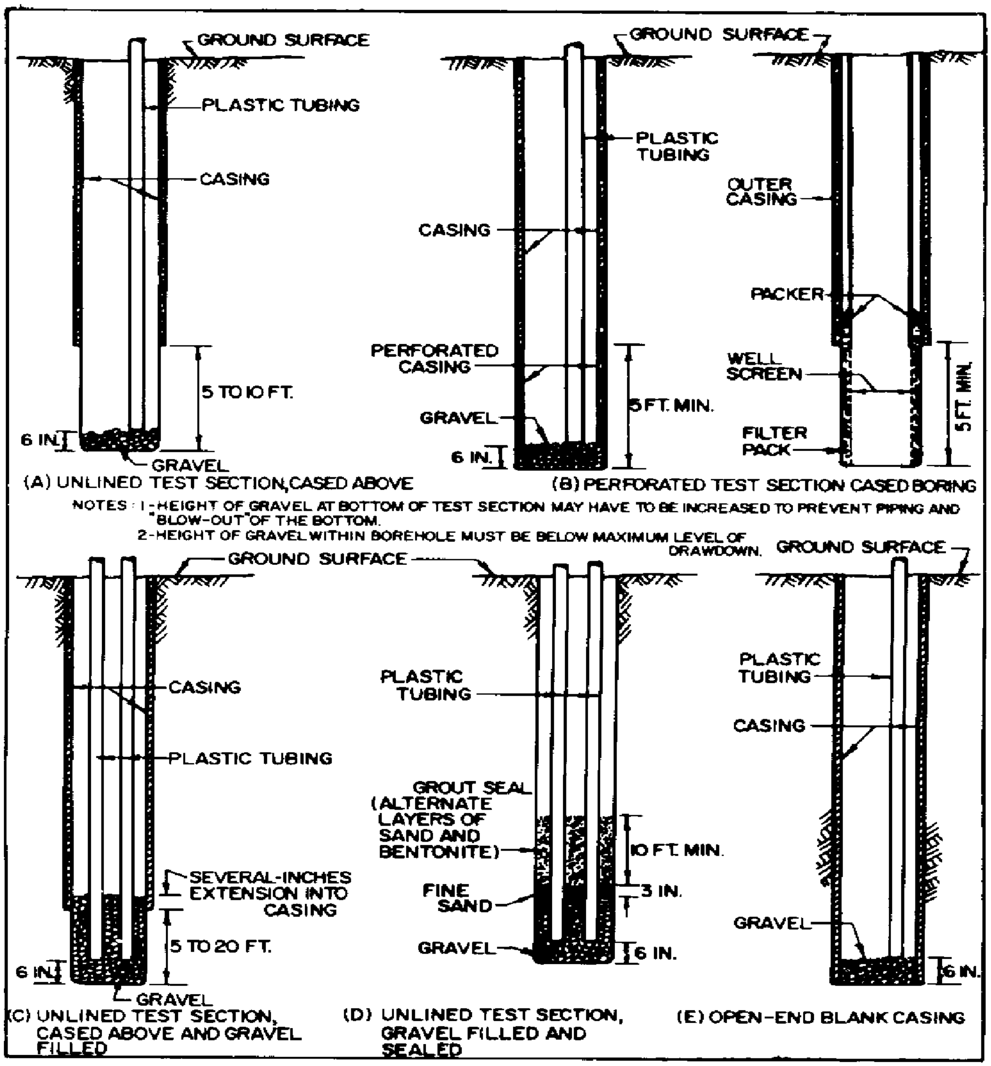


FIGURE 14
Test Zone Isolation Methods

(2) Rising Head Test. In a saturated zone with sufficiently permeable materials, this test is more accurate than a constant or a falling head test. Plugging of the pores by fines or by air bubbles is less apt to occur in a rising head test. In an unsaturated zone, the rising head test is inapplicable.

(3) Falling Head Test. In zones where the flow rates are very high or very low, this test may be more accurate than a constant head test. In an area of unknown permeability the constant head test should be attempted before a falling head test.

(4) Pumping Test. In large scale seepage investigations or groundwater resource studies, the expense of aquifer or pumping tests may be justified as they provide more useful data than any other type of test. Pump tests require a test well, pumping equipment, and lengthy test times. Observation wells are necessary. A vast number of interpretive techniques have been published for special conditions.

(5) Gravity and Pressure Tests. In a boring, gravity and pressure tests are appropriate. The segment of the boring tested is usually 5 to 10 feet, but may be larger. A large number of tests must be conducted to achieve an overall view of the seepage characteristics of the materials. The zone of influence of each test is small, usually a few feet or perhaps a few inches. These methods can detect changes in permeability over relatively short distances in a boring, which conventional pump or aquifer tests cannot. Exploration boring (as opposed to "well") methods are therefore useful in geotechnical investigations where inhomogeneity and anisotropy may be of critical importance. Results from pressure tests using packers in fractured rock may provide an indication of static heads, inflow capacities, and fracture deformation characteristics, but conventional interpretation methods do not give a true permeability in the sense that it is measured in porous media.

c. Percolation Test. The percolation test is used to ascertain the acceptability of a site for septic tank systems and assist in the design of subsurface disposal of residential waste. Generally, the length of time required for percolation test varies with differing soils. Test holes are often kept filled with water for at least four hours, preferably overnight, before the test is conducted. In soils that swell, the soaking period should be at least 24 hours to obtain valid test results.

(1) Type of Test. The percolation test method most commonly used, unless there are specified local requirements, is the test developed by the Robert A. Taft Sanitary Engineering Center as outlined in the Reference 18, Public Health Service Health Manual of Septic Tank Practice, by HUD. A specified hole is dug (generally 2 feet square), or drilled (4 inches minimum) to a depth of the proposed absorption trench, cleaned of loose debris, filled with coarse sand or fine gravel over the bottom 2 inches, and saturated for a specified time. The percolation rate measurement is obtained by filling the hole to a prescribed level (usually 6 inches) and then measuring the drop over a set time limit (usually 30 minutes). In sandy soils the time limit may be only 10 minutes. The percolation rate is used in estimating the required leaching field area as detailed in Reference 18.

5. IN-PLACE DENSITY. In-place soil density can be measured on the surface by displacement methods to obtain volume and weight, and by nuclear density meters. Density at depth can be measured only in certain soils by the drive cylinder (sampling tube) method.

a. Displacement Methods. Direct methods of measuring include sand displacement and water balloon methods. See Reference 19, Evaluation of Relative Density and its Role in Geotechnical Projects Involving Cohesionless Soils, ASTM STP 523. The sand displacement and water balloon methods are the most widely used methods because of their applicability to a wide range of material types and good performance. The sand displacement method (ASTM Standard Dl556, Density of Soil in Place by the Sand Cone Method) is the most frequently used surface test and is the reference test for all other methods. A procedure for the water or rubber balloon method is given in ASTM Standard D2167, Density of Soil in Place by the Rubber Balloon Method.

b. Drive-Cylinder Method. The drive cylinder (ASTM Standard D2937, Density of Soil in Place by the Drive-Cylinder Method) is useful for obtaining subsurface samples from which the density can be ascertained, but it is limited to moist, cohesive soils containing little or no gravel and moist, fine sands that exhibit apparent cohesion.

c. Nuclear Moisture-Density Method. Use ASTM Standard D2922, Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth). Before nuclear density methods are used on the job, results must be compared with density and water contents determined by displacement methods. Based on this comparison, corrections may be required to the factory calibration curves or a new calibration curve may have to be developed. Safety regulations pertaining to the use of nuclear gages are contained in Reference 20, Radiological Safety, U.S. Corps of Engineers ER 385-1-80.

6. DETECTION OF COMBUSTIBLE GASES. Methane and other combustible gases may be present in areas near sanitary landfills, or at sites near or over peat bogs, marshes and swamp deposits. Commercially available indicators are used to detect combustible gases or vapors and sample air in borings above the water table. The detector indicates the concentration of gases as a percentage of the lower explosive limit from 0 to 100 on the gage. The lower explosive limit represents the leanest mixture which will explode when ignited. The gage scale between 60% and 100% is colored red to indicate very dangerous concentrations. If concentrations are judged to be serious, all possibilities of spark generation (e.g., pile driving, especially mandrel driven shells) should be precluded, and a venting system or vented crawl space should be considered. The system could be constructed as follows:

(a) Place a 6-inch layer of crushed stone (3/4-inch size) below the floor slab; the crushed stone should be overlain by a polyethylene vapor barrier.

(b) Install 4-inch diameter perforated pipe in the stone layer below the slab; the top of the pipe should be immediately below the bottom of the slab.

(c) The pipes should be located such that gas rising vertically to the underside of the floor slab does not have to travel more than 25 feet laterally through the stone to reach a pipe.

(d) The pipes can be connected to a single, non-perforated pipe of 6-inch diameter, and vented to the atmosphere at roof level.

Further details on gas detection and venting can be found in References 21, Sanitary Landfill Design Handbook, by Noble, and 22, Process Design Manual, Municipal Sludge Landfills, by the EPA.

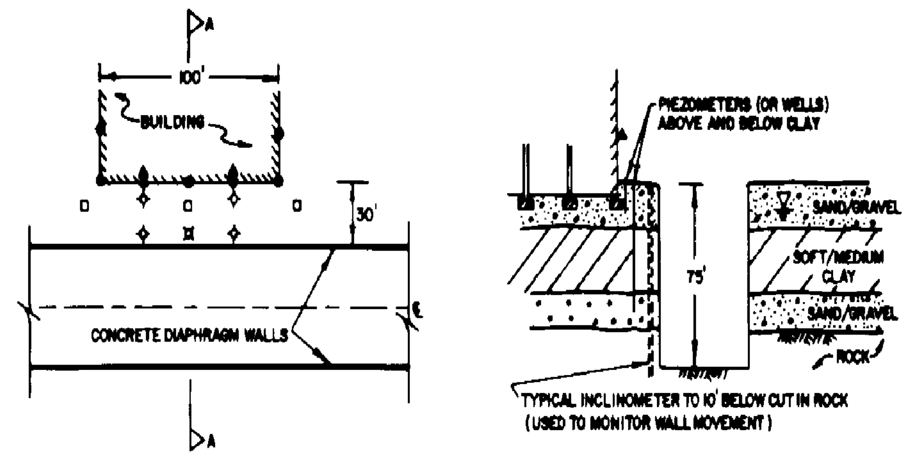
Section 10. FIELD INSTRUMENTATION

1. UTILIZATION. Field instrumentation is used to measure load and displacement and to monitor changes during and after construction. This allows verification of design assumptions and performance monitoring, which could indicate the need for implementation of contingency plans or design changes. For additional guidance on planning and performing geotechnical monitoring see Reference 23, Geotechnical Instrumentation for Monitoring Field Performance, by Dunnicliff. See Reference 24, Equipment for Field Deformation Measurements, by Dunnicliff, for instrumentation devices in current use. See Figure 15 for an example of instrumentation adjacent to a building and diaphragm wall.

a. Survey Technique. The most common uses of optical survey techniques are for the determination of changes in elevation, or lateral displacement. The laser geodimeter provides a significant reduction in time as well as increased accuracy in monitoring of slopes. Survey techniques can be used effectively to monitor surface movement of building and adjacent ground movement of slopes and excavation walls. Figure 15 shows an application of optical surveys.

b. Monitoring of Settlement and Heave. Many devices are available for monitoring settlement and heave, including a number which will permit measurement of the compression of the separate soil layers. Vertical movement can also be measured by remote settlement gages utilizing closed fluid systems, and by extensometers embedded beneath foundations in an incompressible layer. These devices are also well suited to measuring heave. For a more detailed description of field instrumentation equipment see Reference 22, and the latest brochures of geotechnical instrumentation companies.

c. Horizontal and Slope Movements. In addition to conventional surveying techniques, horizontal movement can be measured by horizontal movement gauges, inclinometers, and extensometers. Inclinometers are especially useful for monitoring horizontal soil displacement along the vertical face of a cofferdam or bulkhead, or as in Figure 15, adjacent to an excavation. Tiltmeters can provide very precise measurements of slope changes in soil and rock formations or in structures.



PLAN SECTION A-A

Legend

1. ● δ_v - settlement point at surface
2. ⌖ δ_h - inclinometer (for measurement of subsurface horizontal movements)
3. ⌑ $\delta_h \delta_v$ - inclinometer and multi-point subsurface settlement system
4. □ - piezometer
5. ▲ - tilt meter on building
6. Monitor horizontal movements at the face of the diaphragm wall by optical survey. Monitor settlement of diaphragm wall by optical survey.

FIGURE 15
Example of Instrumentation Adjacent to a Building and Diaphragm Wall

d. Loads and Temperature. See Table 16 (Reference 25, Lateral Support System and Underpinning, Volume II, Design Fundamentals, by Goldberg, et al.) for load and temperature monitoring devices commonly used in walled excavations.

TABLE 16
Load and Temperature Devices in Walled Excavation Elements

Parameter	Instrument	Advantages	Limitations
Load and Stress in Struts, Soldier Piles, Sheet Piles, Wales and Diaphragm Walls.	Mechanical strain gage.	Inexpensive, simple. Easy to install. Minimum damage potential.	Access problems. Many temperature corrections required. Limited accuracy. Readings are subjective.
	Vibrating wire strain gage.	Remote readout. Readout can be automated. Potential for accuracy and reliability. Frequency signal permits data transmission over long distances. Gages can be re-used.	Expensive. Sensitive to temperature, construction damage. Requires substantial skill to install. Risk of zero drift. Risk of corrosion if not hermetically sealed.
	Electrical resistance strain gage.	Inexpensive. Remote readout. Readout can be automated. Potential for accuracy and reliability. Most limitations listed opposite can be overcome if proper techniques are used.	Sensitive to temperature, moisture, cable length change in connections, construction damage. Requires substantial skill to install. Risk of zero drift.
Load in Tieback Anchors.	Telltale load cell.	Inexpensive. Simple. Calibrated in-place.	Access problems. Cannot be used with all proprietary anchor systems.
	Mechanical load cell.	Direct reading. Accurate and reliable. Rugged and durable.	

TABLE 16 (continued)
Load and Temperature Devices in Walled Excavation Elements

Parameter	Instrument	Advantages	Limitations
	Electrical resistance strain gage load cell.	Remote readout. Readout can be automated.	Expensive. Sensitive to temperature, moisture, cable length change.
	Vibrating wire strain gage load cell.	Remote readout. Readout can be automated. Frequency signal permits data transmission over long distances.	Expensive. Sensitive to temperature. Risk of zero drift.
	Photelastic load cell.	Inexpensive.	Limited capacity. Access problems. Requires skill to read.
Temperature	Thermistor	Precise	Delicate, hence susceptible to damage. Sensitive to cable length.
	Thermocouple	Robust. Insensitive to cable length. Available in portable version as "surface pyrometer".	Less precise than thermistor, but premium grade can give $\pm$ 1°F.

REFERENCES

1. Naval Facilities Engineering Command, H.Q., Geotechnical Data Retrieval System, 1980.

2. Glass, C.E. and Slemmons, D.B., Imagery in Earthquake Analysis, Misc. paper S-73-1, State-of-the-Art for Assessing Earthquake Hazards in the United States, USCE, Waterways Experiment Station, Vicksburg, MS., 1978.

3. Way, S.G., Terrain Analysis, A Guide to Site Selection Using Aerial Photographic Interpretation, Dowden, Hutchinson and Ross, Inc., Stroudsburg, PA., 1973.

4. Hvorslev, M.J., Subsurface Exploration and Sampling for Civil Engineering Purposes, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS., 1949.

5. Higginbottom, I.E., The Use of Geophysical Methods in Engineering Geology, Part II, Electrical Resistivity, Magnetic and Gravity Methods, Ground Engineering, Vol. 9, No. 2, 1976.

6. Millet, R.A. and Morehouse, D.C., Bedrock Verification Program for Davis-Besse Nuclear Power Station, Proceedings of the Specialty Conference on Structural Design of Nuclear Plant Facilities, ASCE, 1973.

7. Brenner, R.P. and Phillipson, H.B., Sampling of Residual Soils in Hong Kong, Proceedings of the International Symposium of Soil Sampling, Singapore, 1979.

8. ASTM STP 501, Underwater Soil Sampling, Testing and Construction Control, 1972.

9. Lee, H.J. and Clausner, J.E., Seafloor Soil Sampling and Geotechnical Parameter Determination - Handbook, Civil Engineering Laboratory, Department of the Navy, August, 1979.

10. Marcuson, W.F. III, and Bieganouski, W.A., SPT and Relative Density in Coarse Sands, Journal of Geotechnical Engineering Division, ASCE, Vol. 103, No. GT 11, 1977.

11. Lacroix, Y. and Horn, H.M., Direct Determination and Indirect Evaluation of Relative Density and Earthwork Construction Projects, ASTM STP 523, 1973.

12. Ohsaki, Y., and Iwasaki, R., On Dynamic Shear Moduli and Poisson's Ratios of Soil Deposits, Soils and Foundations Vol. 13, No. 4, 1973.

13. Canadian Geotechnical Society, Properties of Soil and Rock, Canadian Foundation Engineering Manual, Part 1, Canadian Geotechnical Society, 1978.

14. Federal Highway Administration, Guidelines for Cone Penetration Tests Performance and Design, FHWA Report TS-28-209, 1977.

15. Acker Soil Sampling Catalog, Acker Drill Company, Inc., Scranton, PA.

16. Baguelin, F. and Jezequel, J.F., French Self-Boring Pressuremeter, PAF 68-PAF 72 and PAF 76, Report No. FHWA-TS-80-202, Federal Highway Administration, Washington, D.C., 1980.

17. Wroth, C.P., Cambridge In-Situ Probe, PAF 68-PAF 72 and PAF 76, Report No. FHWA-TS-80-202, Federal Highway Administration, Washington, D.C., 1980.

18. HUD, Public Health Service Health Manual of Septic Tank Practice, NTIS PB 218226.

19. ASTM STP 523, Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils, 1972.

20. U.S. Corps of Engineers, Radiological Safety, ER385-1-80.

21. Noble, G., Sanitary Landfill Design Handbook, Technamic Publishing Co., Westport, CT., 1976.

22. United States Environmental Protection Agency (EPA), Process Design Manual, Municipal Sludge Landfills, EPA-625 11-78-010, SW 705, 1978.

23. Dunnicliff, C.J., Geotechnical Instrumentation for Monitoring Field Performance, National Cooperative Highway Research Program, Synthesis of Highway Practice, Transportation Research Board, to be published 1981.

24. Dunnicliff, C.J., Equipment for Field Deformation Measurements, Proceedings of the Fourth Panamerican Conference, SMFE, Vol. II, San Juan, Puerto Rico, January 1973.

25. Goldberg, D.T., Jaworski, W.E., and Gordon, M.D., Lateral Support Systems and Underpinning, Volume II Design Fundamentals, Report No. FHWA-RD-75-129, Federal Highway Administration, 1976.

26. Naval Facilities Engineering Command, Design Manuals (DM) and Publications (P).

DM-5.04	Pavements
P-418	Dewatering and Groundwater Control

Government agencies may obtain copies of design manuals and NAVFAC publications from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, Pennsylvania 19120. Nongovernment agencies and commercial firms may obtain copies from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

CHAPTER 3. LABORATORY TESTING

Section 1. INTRODUCTION

1. SCOPE. This chapter covers laboratory test procedures, typical test properties, and the application of test results to design and construction. Symbols and terms relating to tests and soil properties conform, generally, to definitions given in ASTM Standard D653, Standard Definitions of Terms and Symbols Relating to Soil and Rock Mechanics found in Reference 1, Annual Book of ASTM Standards, by the American Society for Testing and Materials.

2. RELATED CRITERIA. For additional requirements concerning laboratory tests for highway and airfield design, see the following:

Subject	Source
Airfield Pavements......................................	NAVFAC DM-21 Series and DM-21.03
Pavements, Soil Exploration, and Subgrade Testing.......	NAVFAC DM-5.04

3. LABORATORY EQUIPMENT. For lists of laboratory equipment for performance of tests, see Reference 2, Soil Testing for Engineers, by Lambe, Reference 3, The Measurement of Soil Properties in the Triaxial Test, by Bishop and Henkel, and other criteria sources.

4. TEST SELECTION FOR DESIGN. Standard (ASTM) or suggested test procedures, variations that may be appropriate, and type and size of sample are included in Tables 1, 2, 3, and 4. Table 5 lists soil properties determined from such tests, and outlines the application of such properties to design. ASTM procedures are found in Reference 1.

a. Sample Selection. Samples to be tested should be representative, i.e. they should be similar in characteristics to most of the stratum from which they come, or be an average of the range of materials present. If this appears difficult because of variations in the stratum, it may be necessary to consider subdivisions of the stratum for sampling, testing, and design purposes. In general, tests on samples of mixed or stratified material, such as varved clay, should be avoided; usually such results are not indicative of material characteristics; and better data for analysis can be obtained by testing the different materials separately. Undisturbed samples for structural properties tests must be treated with care to avoid disturbance; an "undisturbed" sample found to be disturbed before testing normally should not be tested. Fine-grained cohesive samples naturally moist in the ground should not be allowed to dry before testing, as irreversible changes can occur; organic soils are particularly sensitive. Soils with chemical salts in the pore water may change if water is added, diluting the salt concentration, or if water is removed, concentrating or precipitating the salt. Organic soils require long-term low temperature (60deg.C) drying to avoid severe oxidation (burning) of the organic material.

TABLE 1
Requirements for Index Properties Tests and Testing Standards

Test	Reference for Standard Test procedures[(a)]	Variations from Standard Test Procedures, Sample Requirements	Size or Weight of of Sample for Test[(b)],[(c)]
Moisture content of soil	(1, ASTM D2216)	None. (Test requires unaltered natural moisture content.)	As large as convenient.
Moisture, ash, and organic matter of peat materials	(1, ASTM D2974)	None.	
Dry unit weight	None.	Determine total dry weight of a sample of measured total volume. (Requires undisturbed sample).	As large as convenient.
Specific gravity:			
Material smaller than No. 4 sieve size	(1, ASTM D854)	Volumetric flask preferable; vacuum preferable for de-airing.	25 to 50 for fine-grained soil; 150 gm for coarse-grained soils.
Material larger than No. 4 sieve size	(1, ASTM C127)	None.	500 gm.
Atterberg Limits:		Use fraction passing No. 40 sieve; material should not be dried before testing.	
Liquid limit	(1, ASTM D423)	None.	100 to 500 gm.
Plastic limit	(1, ASTM D424)	Ground glass plate preferable for rolling.	15 to 20 gm.
Shrinkage limit	(4)	In some cases a trimmed specimen of undisturbed material may be used rather than a remolded sample.	30 gm.

TABLE 1 (continued)
Requirements for Index Properties Tests and Testing Standards

Test	Reference for Standard Test procedures[(a)]	Variations from Standard Test Procedures, Sample Requirements	Size or Weight of Sample for Test[(b)],[(c)]
Gradation:			
Sieve analysis	(1, ASTM D422)	Selection of sieves to be utilized may vary for samples of different gradation.	500 gm for soil with grains to 3/8"; to 5,000 m for soil with grains to 3".
Hydrometer analysis	(1, ASTM D422)	Fraction of sample for hydrometer analysis may be that passing No. 200 sieve. For fine-grained soil entire sample may be used. All material must be smaller than No. 10 sieve.	65 gm for fine-grained soil; 115 gm for sandy soil.
Corrosivity:			
Sulphate content	(5)	Several alternative procedures in reference.	soil/water solution prepared see reference.
Chloride content	(5)	Several alternative procedures in reference.	Soil/water solution prepared, see reference.
pH	(1, ASTM Dl293)	Reference is for pH of water. For mostly solid substances, solution made with distilled water and filtrate tested; standard not available.	
Resistivity (laboratory)	None.	Written standard not available. Follow guidelines provided by manufacturers of testing apparatus.	
Resistivity (field)	(6)	In situ test procedure.	

TABLE 1 (continued)
Requirements for Index Properties Tests and Testing Standards

(a) Number in parenthesis indicates Reference number.

(b) Samples for tests may either be disturbed or undisturbed; all samples must be representative and non-segregated; exceptions noted.

(c) Weights of samples for tests on air-dried basis.

TABLE 2
Requirements for Structural Properties

Test	Reference for Suggested Test Procedures[a]	Variations from Suggested Test Procedures	Size or Weight of Sample for Test (undisturbed, remolded, or compacted)
Permeability:			
Constant head procedure for moderately permeable soil	(2),(4)		Sample size depends on max. grain size, 4 cm dia. by 35 cm height for silt and fine sand.
Variable head procedure	(2),(4)	Generally applicable to fine-grained soils.	Similar to constant head sample.
Constant head procedure for coarse-grained soils	(4),(1, ASTM D2434)	Limited to soils containing less than 10% passing No. 200 sieve size. For clean coarse-grained soil the procedure in (4) is preferable.	Sample diameter should be ten times the size of the largest soil particle.
Capillary head	(2)	Capillary head for certain fine-grained soils may have to be determined indirectly.	200 to 250 gm dry weight.
Consolidation:			
Consolidation	(2)	To investigate secondary compression, individual loads may be maintained for more than 24 hours.	Diameter preferably 2-1/2 in or larger. Ratio of diameter to thickness of 3 to 4.

TABLE 2 (continued)
Requirements for Structural Properties

Test	Reference for Suggested Test Procedures[a]	Variations from Suggested Test Procedures	Size or Weight of Sample for Test (undisturbed, remolded, or compacted)
Swell	(7, AASHTO T258)	-	Diameter preferably 2-1/2 in or larger. Ratio of diameter to thickness of 3 to 4.
Collapse potential	(8)	-	2 specimens for each test, with diameter 2-1/2 in or larger. Diameter to height ratio 3 to 4.
Shear Strength:			
Direct shear	(2),(1, ASTM D3080)	Limited to tests on cohesionless soils or to consolidated shear tests on fine-grained soils.	Generally 0.5 in thick, 3 in by 3 in or 4 in by 4 in in plan, or equivalent circular cross section.
Unconfined compression	(2),(1, ASTM D2166)	Alternative procedure given in Reference 4.	Similar to triaxial test samples.

TABLE 2 (continued)
Requirements for Structural Properties

Test	Reference for Suggested Test Procedures[a]	Variations from Suggested Test Procedures	Size or Weight of Sample for Test (undisturbed, remolded, or compacted)
Triaxial compression:			
Unconsolidated - undrained (Q or UU)	(1, ASTM D2850)		Ratio of height to diameter should be less than 3 and greater than 2. Common sizes are: 2.8 in dia., 6.5 in high. Larger sizes are appropriate for gravelly materials to be used in earth embankments.
Consolidated-undrained (R or CU)	(2),(4)	Consolidated-undrained tests may run with or without pore pressure measurements, according to basis for design.	
Consolidated-drained (S or CD)	(2),(4)		
Vane Shear			Block of undisturbed soil at least three times dimensions of vane.

(a) Number in parenthesis indicates Reference number.

TABLE 3
Requirements for Dynamic Tests

Test	Reference for Test Procedure[a],[b]	Variations from Standard Test Procedure	Size or Weight of Sample for Test
Cyclic Loading			
Triaxial compression	(9)		Same as for structural properties triaxial.
Simple shear	(9)		
Torsional shear	(10)	Can use hollow specimen.	
Resonant Column	(10) & (11)	Can use hollow specimen.	Same as for structural properties triaxial; length sometimes greater.
Ultrasonic pulse			
Soil	(12)		Same as for structural properties triaxial.
Rock	(1, ASTM D2845)		Prism, length less than 5 times lateral dimension; lateral dimension at least 5 times length of compression wave.

(a) Number in parenthesis indicates Reference number.

(b) Except for the ultrasonic pulse test on rock, there are no recognized standard procedures for dynamic testing. References are to descriptions of tests and test requirements by recognized authorities in those areas.

TABLE 4
Requirements for Compacted Samples Tests

Test	Reference for Standard Test Procedures[(a),(b)]	Variations from Standard Test Procedures	Size or Weight of Sample for Test[(c)]
Moisture-density relations:			
Standard Proctor 5-1/2 lb. hammer, 12 in. drop	(1, ASTM D698)	Preferable not to reuse samples for successive compaction determinations.	Each determination (typically 4 or 5 determinations per test): Method A: 6 lbs Method B: 14 lbs Method C: 10 lbs Method D: 22 lbs
Modified Proctor 10 lb. hammer, 18 in. drop	(1, ASTM D1557)	Preferable not to reuse samples for successive compaction determinations.	Method A: 7 lbs Method B: 16 lbs Method C: 12 lbs Method D: 25 lbs
Maximum and Minimum Densities of Cohesionless Soils	(1, ASTM D2049), (4)		Varies from 10 to 130 lbs depending on max. grain size.
California Bearing Ratio	(1, ASTM D1883)	Compaction energy other than that for Modified Proctor may be utilized.	Each determination requires 15 to 25 lbs depending on gradation.
Resistance R-value	(1, ASTM D2844)		10 - 15 lbs depending on gradation.

TABLE 4 (continued)
Requirements for Compacted Samples Tests

Test	Reference for Standard Test Procedures[a],[b]	Variations from Standard Test Procedures	Size or Weight of Sample for Test[c]
Expansion Pressure	(7, AASHTO T190)	Alternatively, testing procedures of Table 2 may be utilized.	10 - 15 lbs depending on gradation.
Permeability and compression	(13)	Best suited for coarse-grained soils. Alternatively, testing procedures of Table 2 may be utilized.	15 lbs of material passing No. 4 sieve size.

(a) Number in parenthesis indicates Reference number.

(b) For other sources of standard test procedures, see Table 1.

(c) Weight of samples for tests given on air-dried basis.

TABLE 5
Soil Properties for Analysis and Design

Property	Symbol	Unit[a]	How Obtained	Direct Applications
Volume-weight Characteristics[b]				
Moisture Content	w	D	Direct from test	Classification and volume-weight relations.
Unit Weights	γ	FL^{-3}	Directly from test or from volume-weight relations	Classification and pressure computations.
Porosity	n	D	Computed from volume-weight relations	Parameters used to represent relative volume of voids with respect to total volume of soil or volume of solids.
Void Ratio	e	D	Computed from volume-weight relations	
Specific Gravity	G	D	Directly from test	Volume computations.
Plasticity Characteristics:				
Liquid Limit	LL	D	Directly from test	Classification and properties correlation.
Plastic Limit	PL	D	Directly from test	
Plasticity Index	PI	D	LL-PL	

TABLE 5 (continued)
Soil Properties for Analysis and Design

Property	Symbol	Unit[(a)]	How Obtained	Direct Applications
Shrinkage limit	SL	D	Directly from test.	Classification and computation of swell.
Shrinkage index	SI	D	PL-SL	
Activity	A_c	D	$\frac{PI}{\% <2 \text{ microns}}$	Identification of clay mineral.
Liquidity index	LI	D	$\frac{w - PL}{PI}$	Estimating degree of preconsolidation, and soil consistency.
Gradation Characteristics:				
Effective diameter	D_{10}	L	From grain size curve.	
Percent grain size	D_{30} D_{60} D_{85}	L	From grain size curve.	Classification, estimating permeability and unit weight, filter design, grout selection, and evaluating potential frost heave and liquefaction.
Coefficient of uniformity	C_u	D	$\frac{D_{60}}{D_{10}}$	
Coefficient of curvature	C_z	D	$\frac{(D_{30})^2}{(D_{10})x(D_{60})}$	
Clay size fraction	-	D	From grain size curve, % finer than 0.002 mm.	

TABLE 5 (continued)
Soil Properties for Analysis and Design

Property	Symbol	Unit(a)	How Obtained	Direct Applications
Drainage Characteristics:				
Coefficient of permeability	k	LT^{-1}	Directly from permeability test or computed from consolidation test data.	Drainage, seepage, and consolidation analysis.
Capillary head	h_c	L	Directly from test.	Drainage and drawdown analysis.
Effective porosity	n_c	D	Directly from test for volume of drainable water.	
Consolidation Characteristics:				
Coefficient of compressibility	a_v	L^2F^{-1}	Determined from natural plot of e vs. p curve.	Computation of ultimate settlement or heave in consolidation analysis.
Coefficient of volume compressibility	m_v	L^2F^{-1}	$\frac{a_v}{1 + e}$	
Compression index	C_c	D	Determined from e vs. log p curve.	Computation of ultimate settlement or swell in consolidation analysis.
Recompression index	C_r	D		
Swelling index	C_s	D		

TABLE 5 (continued)
Soil Properties for Analysis and Design

Property	Symbol	Unit[a]	How Obtained	Direct Applications
Coefficient of secondary compression	C	D	Determined from semilog time-consolidation curve.	Computation of time rate of settlement.
Coefficient of consolidation	c_v	L^2T^{-1}		
Preconsolidation pressure	P_c	FL^{-2}	Estimate from e vs. log p curve.	Settlement analysis.
Overconsolidation ratio	OCR	D	$\frac{P_c}{P_o}$	Basis for normalizing behavior of clay.
Shear Strength Characteristics:				
Apparent angle of shearing resistance	Ø	A	Determined from Mohr circle plot of shear test data for total stress.	
Cohesion intercept	c	FL^{-2}		
Effective angle of shearing resistance	Ø'	A	Determined from Mohr circle plot of effective stress shear test data (drained tests with pore pressure measurements).	
Effective cohesion	c'	FL^{-2}		

TABLE 5 (continued)
Soil Properties for Analysis and Design

Property	Symbol	Unit[a]	How Obtained	Direct Applications
Unconfined compressive strength	q_u	FL^{-2}	Directly from test.	Analysis of stability, load carrying capacity of foundations, lateral earth load.
Undrained shear strength	s_u	FL^{-2}		
Sensitivity	S_t	D	$\frac{\text{undisturbed strength}}{\text{remolded strength}}$	
Modulus of elasticity or Young's modulus	E_s	FL^{-2}	Determined from stress-strain curve or dynamic test.	Computation of elastic settlement or rebound.
Compaction Characteristics:				
Maximum dry unit weight	γ_{max}	FL^{-3}	Determined from moisture-dry unit weight curve.	Compaction criterion.
Optimum moisture content	OMC	D		
Maximum and minimum density of cohesionless soils	$\gamma_{d\ max}$ $\gamma_{d\ min}$	FL^{-3} FL^{-3}	Directly from test.	
Characteristics of Compacted Samples:				
Percent compaction	-	D	$\frac{\gamma}{\gamma_{max}}$	Compaction control, properties correlation.
Needle penetration resistance	P_r	FL^{-2}	Directly from test.	Moisture control of compaction.

TABLE 5 (continued)
Soil Properties for Analysis and Design

Property	Symbol	Unit[a]	How Obtained	Direct Applications
Relative density	D_r	D	Determined from results of max. and min. density tests.	Compaction control, properties correlation, liquefaction studies.
California Bearing Ratio	CBR	D	Directly from test.	Pavement design, compaction control.
Dynamic Characteristics:				
Shear modulus	G	FL^{-2}	Determined from resonant column, cyclic simple shear, ultrasonic pulse, or dynamic triaxial tests.	Analysis of foundation and soil behavior under dynamic loading.
Damping ratios	-	D	Determined from resonant column test, dynamic triaxial, or cyclic simple shear test.	
Rod (longitudinal)	D_L			
Shear (torsional)	D_T			
Resonant frequency	-	T^{-1}	Determined from resonant column test.	
longitudinal	f_L			
torsional	f_T			

(a) Units F = Force or weight; L = Length; T = Time; D = Dimensionless; A = Angular Measure

(b) For complete list of volume - weight relationships, see Table 6

b. Index Properties Tests. Index properties are used to classify soils, to group soils in major strata, to obtain estimates of structural properties (see correlations in this Chapter), and to correlate the results of structural properties tests on one portion of a stratum with other portions of that stratum or other similar deposits where only index test data are available. Procedures for most index tests are standardized (Table 1). Either representative disturbed or undisturbed samples are utilized. Tests are assigned after review of boring data and visual identification of samples recovered. For a simple project with 4 to 6 borings, at least 3 gradation and/or Atterberg tests should be made per significant stratum (5 to 15 feet thick). For complex soil conditions, thick strata, or larger sites with more borings, additional tests should be made. Moisture content tests should be made liberally on samples of fine-grained soil. In general, the test program should be planned so that soil properties and their variation can be defined adequately for the lateral and vertical extent of the project concerned.

c. Tests for Corrosivity. The likelihood of soil adversely affecting foundation elements or utilities (concrete and metal elements) can be evaluated on a preliminary basis from the results of the tests referenced in Table 1. The tests should be run on samples of soil which will be in contact with the foundations and/or utilities in question; typically these will be only near-surface materials. For a simple project with uniform conditions, three sets of tests may be adequate. Usually the chemical tests are run only if there is reason to suspect the presence of those ions. (See DM-5.7 for application of test results and possible mitigating measures.)

d. Structural Properties Tests. These must be planned for particular design problems. Rigid standardization of test programs is inappropriate. Perform tests only on undisturbed samples obtained as specified in Chapter 2 or on compacted specimens prepared by standard procedures. In certain cases, completely remolded samples are utilized to estimate the effect of disturance. Plan tests to determine typical properties of major strata rather than arbitrarily distributing tests in proportion to the number of undisturbed samples obtained. A limited number of high quality tests on carefully selected representative undisturbed samples is preferred. In general, selecting design values requires at least three test values for simple situations of limited areal extent; larger and more complex conditions require several times these numbers.

Where instantaneous deformation characteristics of soils are to be evaluated, constitutive relationships of the materials in question must also be established. For initial estimates of Young's modulus, E_s, see Chapter 5, and for K_o value, see DM-7.2, Chapter 3.

e. Dynamic Tests. Dynamic testing of soil and rock involves three ranges: low frequency (generally less than 10 hertz) cyclic testing, resonant column high frequency testing, and ultrasonic pulse testing. The dynamic tests are used to evaluate foundation support characteristics under repeated loadings such as a drop forge, traffic, or earthquake; a primary concern is often liquefaction. Young's modulus (E_s), shear modulus (G), and damping characteristics are determined by cyclic triaxial and simple shear tests. Resonant column can be used to determine E_s, G, and damping.

From the resonant frequency of the material in longitudinal, transverse, and torsional modes, Poisson's ratio ([upsilon]) can be computed from test data. Foundation response to dynamic loading, and the effect of wave energy on its surroundings is studied in the light of these test results. The ultrasonic pulse test also evaluates the two moduli and Poisson's ratio, but the test results are more reliable for rocks than for soils.

Dynamic tests can be run on undisturbed or compacted samples, but should be run only if the particular project really requires them. The number of tests depends on project circumstances. Estimates of dynamic parameters can be obtained from correlations with other properties (see references in Section 6 of this chapter).

f. Compaction Tests. In prospecting for borrow materials, index tests or compaction tests may be required in a number proportional to the volume of borrow involved or the number of samples obtained. Structural properties tests are assigned after borrow materials have been grouped in major categories,by index and compaction properties. Select samples for structural tests to represent the main soil groups and probable compacted condition. At least one compaction or relative density test is required for each significantly different material (based on gradation or plasticity). Numbers of other tests depend on project requirements.

g. Typical Test Properties. Various correlations between index and structural properties are available showing the probable range of test values and relation of parameters. In testing for structural properties, correlations can be used to extend results to similar soils for which index values only are available. Correlations are of varying quality, expressed by standard deviation, which is the range above and below the average trend, within which about two-thirds of all values occur. These relationships are useful in preliminary analyses but must not supplant careful tests of structural properties. The relationships should never be applied in final analyses without verification by tests of the particular material concerned.

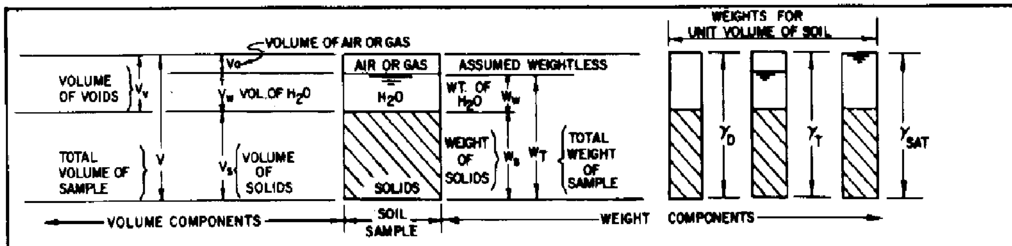
Section 2. INDEX PROPERTIES TESTS

1. MOISTURE CONTENT, UNIT WEIGHT, SPECIFIC GRAVITY. Index properties tests are used to compute soil volume and weight components (Table 6). Ordinarily, determine moisture content for all the representative samples (disturbed or undisturbed) for classification and grouping of materials in principal strata. See Table 1 for test standards.

a. Unsaturated Samples. Measure moisture content, dry weight, specific gravity, and total volume of specimen to compute volume-weight relationships.

b. Saturated Samples. If moisture content and dry weight are measured, all volume-weight parameters may be computed by assuming a specific gravity. If moisture content and specific gravity are measured, all volume-weight parameters may be computed directly. Volume-weight of fine-grained soils below the water table may be determined with sufficient accuracy by assuming saturation.

TABLE 6
Volume and Weight Relationships



	PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS			
VOLUME COMPONENTS	V_s VOLUME OF SOLIDS	$\frac{W_s}{G\gamma_w}$		$V-(V_a+V_w)$	$V(1-n)$	$\frac{V}{(1+e)}$	$\frac{V_v}{e}$
	V_w VOLUME OF WATER	$\frac{W_w}{\gamma_w}$		V_v-V_a	SV_v	$\frac{SVe}{(1+e)}$	$SV_s e$
	V_a VOLUME OF AIR OR GAS	ZERO	$V-(V_s+V_w)$	V_v-V_w	$(1-S)V_v$	$\frac{(1-S)V_e}{(1+e)}$	$(1-S)\ V_s\ e$
	V_v VOLUME OF VOIDS	$\frac{W_w}{\gamma_w}$	$V-\frac{W_s}{G\gamma_w}$	$V-V_s$	$\frac{V_s n}{1-n}$	$\frac{V_e}{(1+e)}$	$V_s\ e$
	V TOTAL VOLUME OF SAMPLE	V_s+V_w	MEASURED	$V_s+V_a+V_w$	$\frac{V_s}{1-n}$	$V_s(1+e)$	$\frac{V_v(1+e)}{e}$
	n POROSITY	$\frac{V_v}{V}$		$1-\frac{V_s}{V}$	$1-\frac{W_s}{GV\gamma_w}$	$\frac{e}{1+e}$	
	e VOID RATIO	$\frac{V_v}{V_s}$		$\frac{V}{V_s}-1$	$\frac{GV\gamma_w}{W_s}-1$	$\frac{W_w G}{W_s S}$	$\frac{n}{1-n}$ | $\frac{wG}{S}$

TABLE 6 (continued)
Volume and Weight Relationships

		PROPERTY	SATURATED SAMPLE (W_s, W_w, G, ARE KNOWN)	UNSATURATED SAMPLE (W_s, W_w, G, V, ARE KNOWN)	SUPPLEMENTARY FORMULAS RELATING MEASURED AND COMPUTED FACTORS			
WEIGHTS FOR SPECIFIC SAMPLE	W_s	WEIGHT OF SOLIDS	MEASURED		$\frac{W_T}{(1+W)}$	$GV\gamma_w(1-n)$	$\frac{W_w G}{eS}$	
	W_w	WEIGHT OF WATER	MEASURED		wW_s	$S\gamma_w V_v$	$\frac{eW_s S}{G}$	
	W_t	TOTAL WEIGHT OF SAMPLE	$W_s + W_w$		$W_s(1+w)$			
WEIGHTS FOR SAMPLE OF UNIT VOLUME	γ_D	DRY UNIT WEIGHT	$\frac{W_s}{V_s + V_w}$	$\frac{W_s}{V}$	$\frac{W_t}{V(1+w)}$	$\frac{G\gamma_w}{(1+e)}$	$\frac{G\gamma_w}{1+wG/S}$	
	γ_T	WET UNIT WEIGHT	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+W_w}{V}$	$\frac{W_T}{V}$	$\frac{(G+Se)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w/S+1/G}$	
	γ_{SAT}	SATURATED UNIT WEIGHT	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+V_v\gamma_w}{V}$	$\frac{W_s}{V}+\left(\frac{e}{1+e}\right)\gamma_w$	$\frac{(G+e)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w+1/G}$	
	γ_{SUB}	SUBMERGED (BUOYANT) UNIT WEIGHT	$\gamma_{SAT} - \gamma_w$		$\frac{W_s}{V}-\left(\frac{1}{1+e}\right)\gamma_w$	$\left(\frac{G+e}{1+e}-1\right)\gamma_w$	$\left(\frac{1-1/G}{w+1/G}\right)\gamma_w$	
COMBINED RELATIONS	w	MOISTURE CONTENT	$\frac{W_w}{W_s}$		$\frac{W_t}{W_s}-1$	$\frac{Se}{G}$	$S\left[\frac{\gamma_w}{\gamma_D}-\frac{1}{G}\right]$	
	S	DEGREE OF SATURATION	1.00	$\frac{V_w}{V_v}$	$\frac{W_w}{V_v\gamma_w}$	$\frac{wG}{e}$	$\frac{w}{\left[\frac{\gamma_w}{\gamma_D}-\frac{1}{G}\right]}$	
	G	SPECIFIC GRAVITY	$\frac{W_s}{V_s\gamma_w}$		$\frac{Se}{w}$			

2. GRADATION. In addition to their use in classification, grain-size analyses may be applied to seepage and drainage problems, filter and grout design, and evaluation of frost heave. See Table 1 for test standards.

a. Grain-Size Parameters. Coefficient of uniformity, C_u, and coefficient of curvature, C_z, are computed from D_{60}, D_{30}, and D_{10}, which are particle size diameter corresponding respectively to 60%, 30%, and 10% passing on the cumulative particle size distribution curves. C_u and C_z indicate the relative broadness or narrowness of gradation. D_{10} is an approximate measure of the size of the void spaces in coarse-grained soils. See Chapter 1.

b. Testing Program. Gradations of a large number of samples usually are not required for identification. Samples should be grouped in principal strata by visual classification before performing grain-size analyses on specimens of major strata.

3. ATTERBERG LIMITS. For classification of the fine-grained soils by Atterberg Limits, see Chapter 1. In addition to their use in soil classification, Atterberg Limits also are indicators of structural properties, as shown in the correlations in this chapter. Atterberg Limit tests should be performed discriminately, and should be reserved for representative samples selected after evaluating subsoil pattern. Determine Atterberg Limits of each consolidation test sample and each set of samples grouped for triaxial shear tests. For selected borings, determine Atterberg Limits on samples at regular vertical intervals for a profile of Limits and corresponding natural water content. See Table 1 for test standards.

Section 3. PERMEABILITY TESTS

1. APPLICATIONS. Permeability coefficient is used to compute the quantity and rate of water flow through soils in drainage and seepage analysis. Laboratory tests are appropriate for undisturbed samples of fine-grained materials and compacted materials in dams, filters, or drainage structures. See Table 2 for test standards and recommended procedures.

a. Fine-Grained Soils. Permeability of fine-grained soils (undisturbed or compacted) generally is computed from consolidation test data or by direct measurement on consolidation or triaxial shear specimens. For soils with permeability less than 10^{-6} cm/sec, a sealant must be used between the specimen and the wall of the permeameter.

b. Sand Drain Design. Sand drain design may require complete permeability data for soils to be stabilized, including determination of permeabilities in both vertical and horizontal direction.

c. Field Permeability Tests. The secondary structure of in situ soils, stratification, and cracks have a great influence on the permeability. Results of laboratory tests should be interpreted with this in mind, and field permeability tests (Chapter 2) should be performed where warranted.

2. TYPICAL VALUES. Coefficient of permeability is a property highly sensitive to sample disturbance, and shows a wide range of variation due to differences in structural characteristics. See Reference 14, Soil Mechanics in Engineering Practice, Terzaghi and Peck, for correlations of permeability with soil type. Permeability of clean, coarse-grained samples is related to D_{10} size (Figure 1).

Section 4. CONSOLIDATION TESTS

1. UTILIZATION. One-dimensional consolidation tests with complete lateral confinement are used to determine total compression of fine-grained soil under an applied load and the time rate of compression caused by gradual volume decrease that accompanies the squeezing of pore water from the soil. See Figure 2 for test relationships.

2. TESTING PROGRAM. Consolidation tests require undisturbed samples of highest quality. Select samples representative of principal compressible strata. Determination of consolidation characteristics of a stratum requires from two to about eight tests, depending on the complexity of conditions. Select loading program to bracket anticipated field loading conditions.

a. Incremental Loading (IL) With Stress Control. Ordinarily, apply loads starting at 1/4 tsf and increase them by doubling 1/2, 1, 2, 4, 8, etc., tsf. For soils with pronounced swelling tendency, it may be necessary to rapidly increase loading to 1/2 tsf or higher, perhaps to overburden pressure, to prevent initial swell. For soft, normally consolidated soils, start loading at 1/16 or 1/32 tsf and increase loads by doubling the previous value. (See Reference 2.) To establish the reconsolidation index C_r, and swelling index C_s, include an unload-reload cycle, after P_c has been reached. Unload must be to 1/8 the existing load, or preferably less. Reloads should be applied in the same manner as for the initial curve.

b. Constant Rate of Strain (CRS). The specimen is subjected to a constantly changing load while maintaining a constant rate of strain. Pore pressure is continuously monitored to ensure that the primary consolidation is completed at the applied strain rate. These tests can be performed in shorter time than IL tests and yield more accurate values of preconsolidation pressure P_c. Coefficient of consolidation c_v values can be determined for very small load increments, but the test equipment is more complicated and requires that estimates of strain rate and P_c be made prior to the start of the test. See Reference 15, Consolidation at Constant Rate of Strain, by Wissa, et al., for guidance.

c. Gradient Controlled Test (GC). Drainage is permitted at the upper porous stone while pore pressure is measured at the lower porous stone. A loading control system regulates the application of load so that a predetermined hydrostatic excess pressure is maintained at the bottom of the specimen. This method as well as CRS has similar advantages over IL, but does not require a prior estimate of strain rate. However, the equipment is more complex than for CSR. See Reference 16, New Concepts in Consolidation and Settlement Analysis, by Lowe, for guidance.

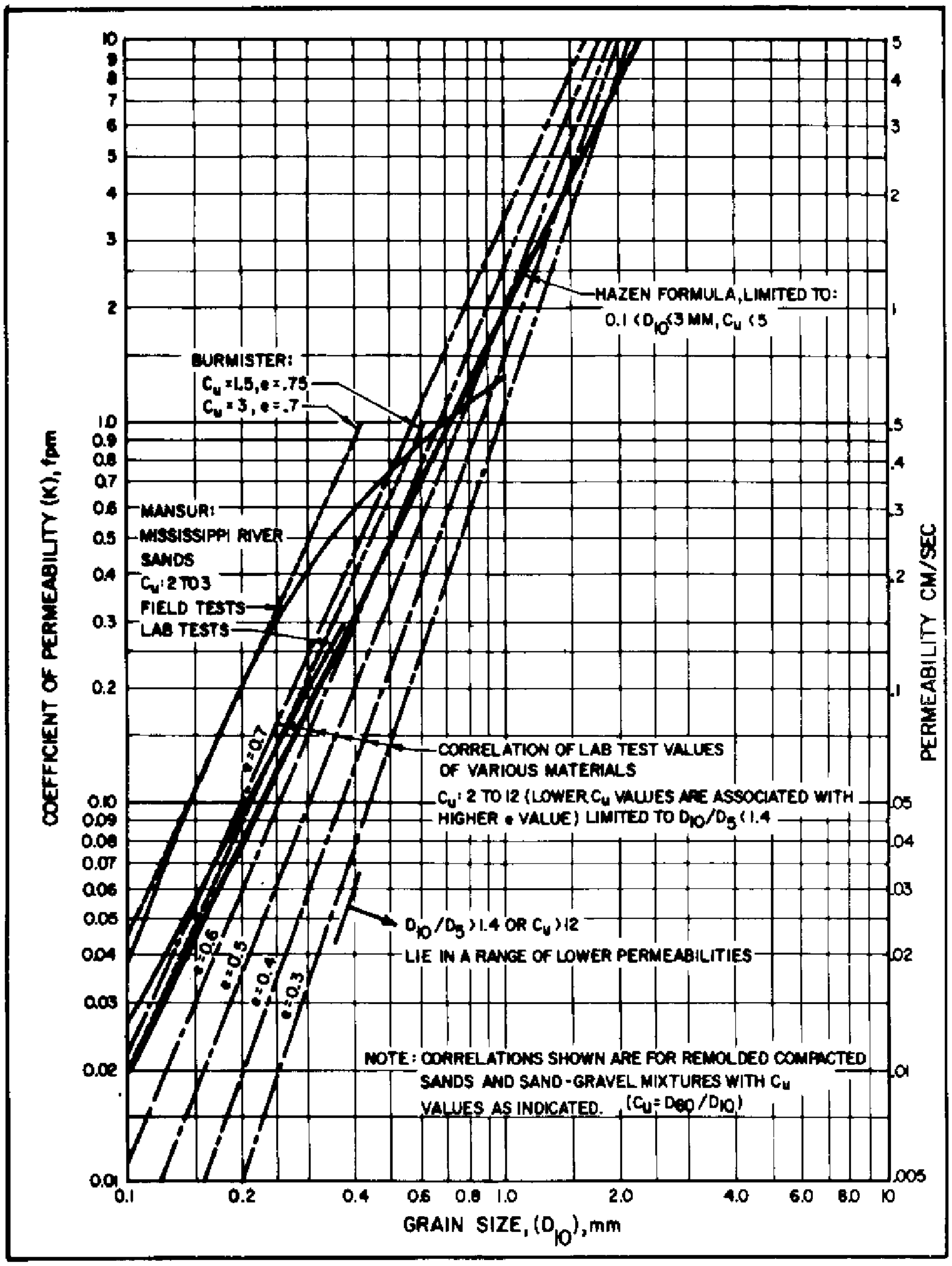


FIGURE 1
Permeability of Sands and Sand-Gravel Mixtures

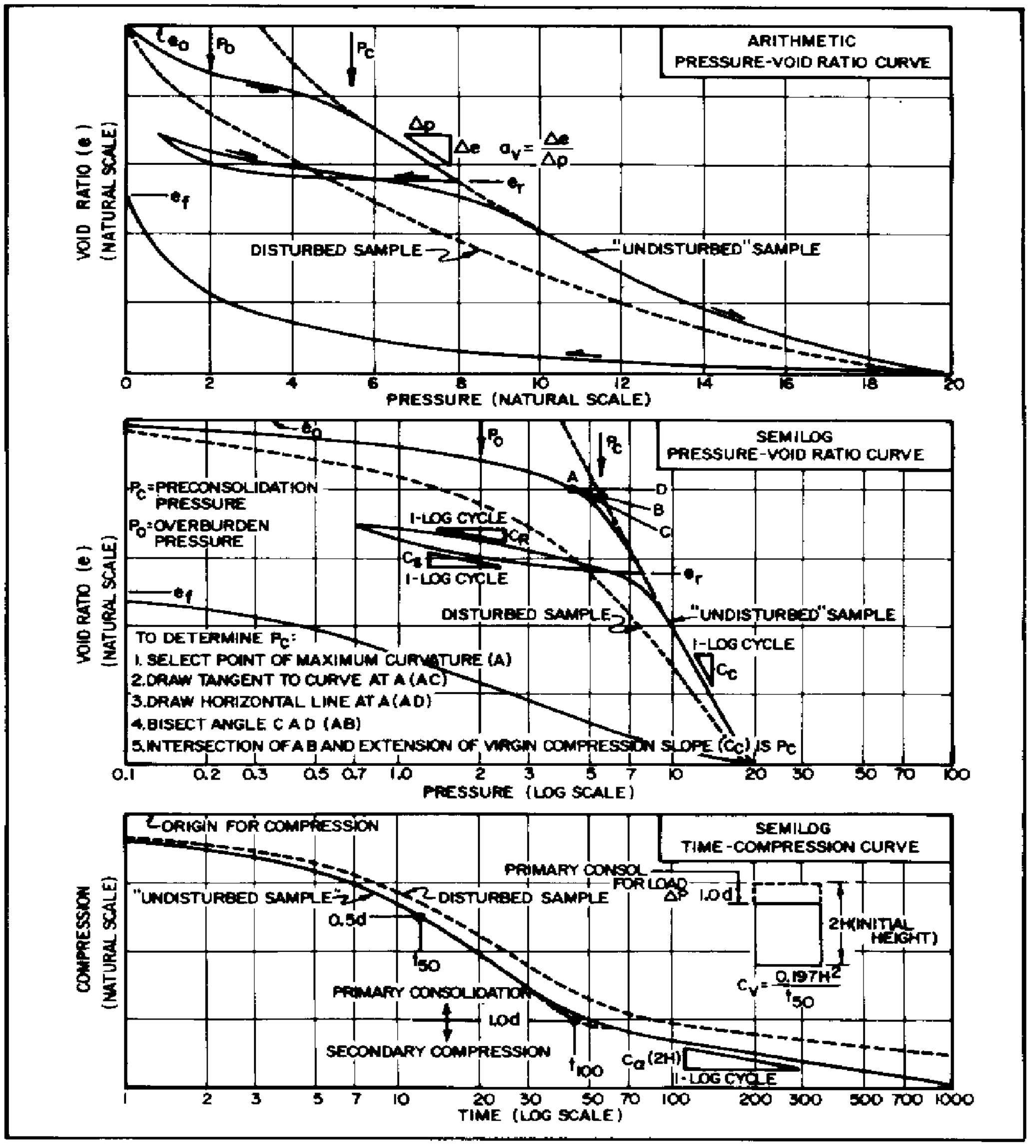


FIGURE 2
Consolidation Test Relationships

3. PRECONSOLIDATION PRESSURE. This pressure value, P_c, forms the boundary between recompression and virgin compression ranges and is approximately the maximum normal effective stress to which the material in situ has been consolidated by a previous loading. Desicccation produces a similar effect. The preconsolidation pressure cannot be determined precisely, but can be estimated from consolidation tests on high quality undisturbed samples.

a. Graphical Determination. Estimate preconsolidation pressure from semilogarithmic pressure-void ratio curve using the procedure given in the central panel of Figure 2. Alternative methods are given in Reference 17, Foundation Engineering, by Leonards, and Reference 18, The Undisturbed Consolidation of Clay, by Schmertmann. Maximum test pressures should exceed preconsolidation by an amount sufficient to define the slope of virgin compression. Generally, this requires application of three or more load increments exceeding the preconsolidation value.

b. Approximate Values. See Figure 3 for a relationship between preconsolidation pressure and liquidity index. For samples with natural moisture at the liquid limit (liquidity index of 1), preconsolidation ranges between about 0.1 and 0.8 tsf depending on soil sensitivity. For natural moisture at the plastic limit (liquidity index equal to zero), preconsolidation ranges from about 12 to 25 tsf.

Alternately estimate: $P_c = \dfrac{q_u/2}{0.11 + 0.0037\,PI}$ in which q_u is the unconfined compressive strength, and PI is the soil plasticity index.

4. VIRGIN COMPRESSION. Virgin compression is deformation caused by loading in the range of pressures exceeding that to which the sample has been subjected in the past.

a. Compression Index. The semilogarithmic, pressure-void ratio curve is roughly linear in the virgin range. The semilogarithmic, straight line slope for virgin compression is expressed by the compression index C_c. (See Figure 2.)

b. Approximate Values. The compression index of silts, clays, and organic soils has been correlated with the natural water content, initial void ratio and the liquid limit. Approximate correlations are given in Chapter 5. The approximate values of C_c for uniform sands in the load range of 1 to 4 tsf may vary from 0.05 to 0.06 (loose condition), and from 0.02 to 0.03 (dense condition).

5. RECOMPRESSION AND SWELL. Depending on the magnitude of preconsolidation, pressures applied by new construction may lie partly or wholly in the recompression range. If the load is decreased by excavation, fine-grained soil will undergo a volumetric expansion in the stress range below preconsolidation.

a. Swelling Index. The slope of straight-line rebound of the semilogarithmic pressure-void ratio curve is defined by C_s (see Figure 2). The swelling index is generally one-fifth to one-tenth of the compression index except for soils with very high swell potential. For typical values of C_s, see Chapter 5.

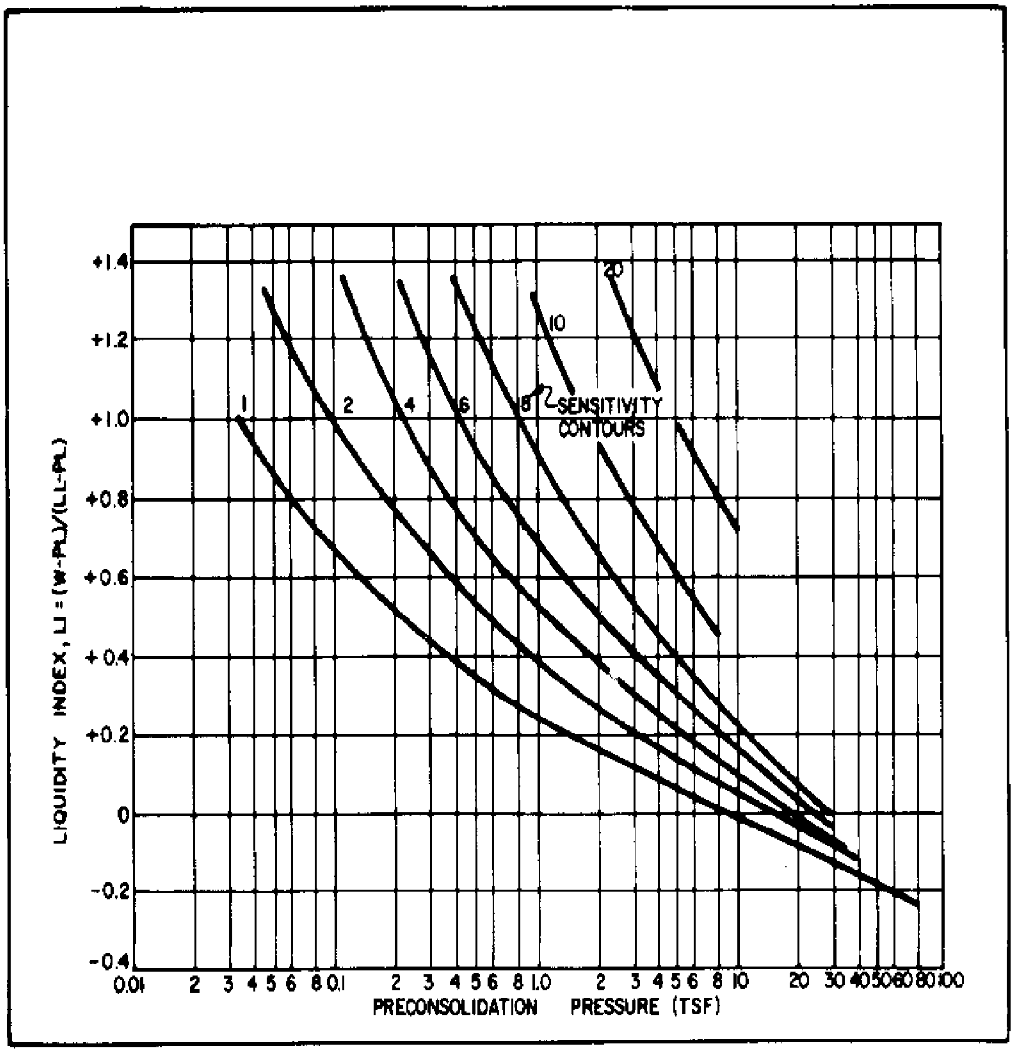


FIGURE 3
Preconsolidation Pressure vs. Liquidity Index

b. Recompression Index. The slope of the straight line in the recompression range of the semilogarithmic pressure-void ratio curve is defined by C+r,, where C+r, is equal to or less than C+s,. (See Figure 2).

6. COMPRESSION OF COLLAPSIBLE SOILS. Such soils require a special test for determining their collapse potential. See Chapter 1 for test details.

7. COEFFICIENT OF CONSOLIDATION (c+v,). Those soil properties that control the drainage rate of pore water during consolidation are combined in the coefficient of consolidation.

a. Determination. Compute c+v, from the semilogarithmic time-compression curve for a given load increment (bottom panel of Figure 2). Correct the origin for compression for the effect of air or gas in void spaces by the procedure given in Reference 2.

b. Approximate Values. Figure 4 may be used to determine approximate values of c+v,.

8. SECONDARY COMPRESSION. After completion of primary consolidation under a specific load, the semilogarithmic time-compression curve continues approximately as a straight line. This is termed secondary compression (Figure 2). It occurs when the rate of compression is no longer primarily controlled by the rate at which pore water can escape; there are no excess pore pressures remaining.

a. Organic Materials. In organic materials, secondary compression may dominate the time-compression curve, accounting for more than one-half of the total compression, or even obliterating the change in slope used to establish the limit of primary compression.

b. Approximate Values. The coefficient of secondary compression C+[alpha], is a ratio of decrease in sample height to initial sample height for one cycle of time on log scale. See bottom panel of Figure 4 for typical values.

9. SAMPLE DISTURBANCE. Sample disturbance seriously affects the values obtained from consolidation tests as shown in Figure 2 and below.

a. Void Ratio. Sample disturbance lowers the void ratio reached under any applied pressure and makes the location of the preconsolidation stress less distinct.

b. Preconsolidation Pressure. Sample disturbance tends to lower the compression index (C+c,) and the preconsolidation pressure (P+c,) obtained from the test curve.

c. Recompression and Swelling. Sample disturbance increases the recompression and swelling indices.

d. Coefficient of Consolidation. Sample disturbance decreases coefficient of consolidation for both recompression and virgin compression. For an undisturbed sample, c+v, usually decreases abruptly at preconsolidation stress. This trend is not present in badly disturbed samples.

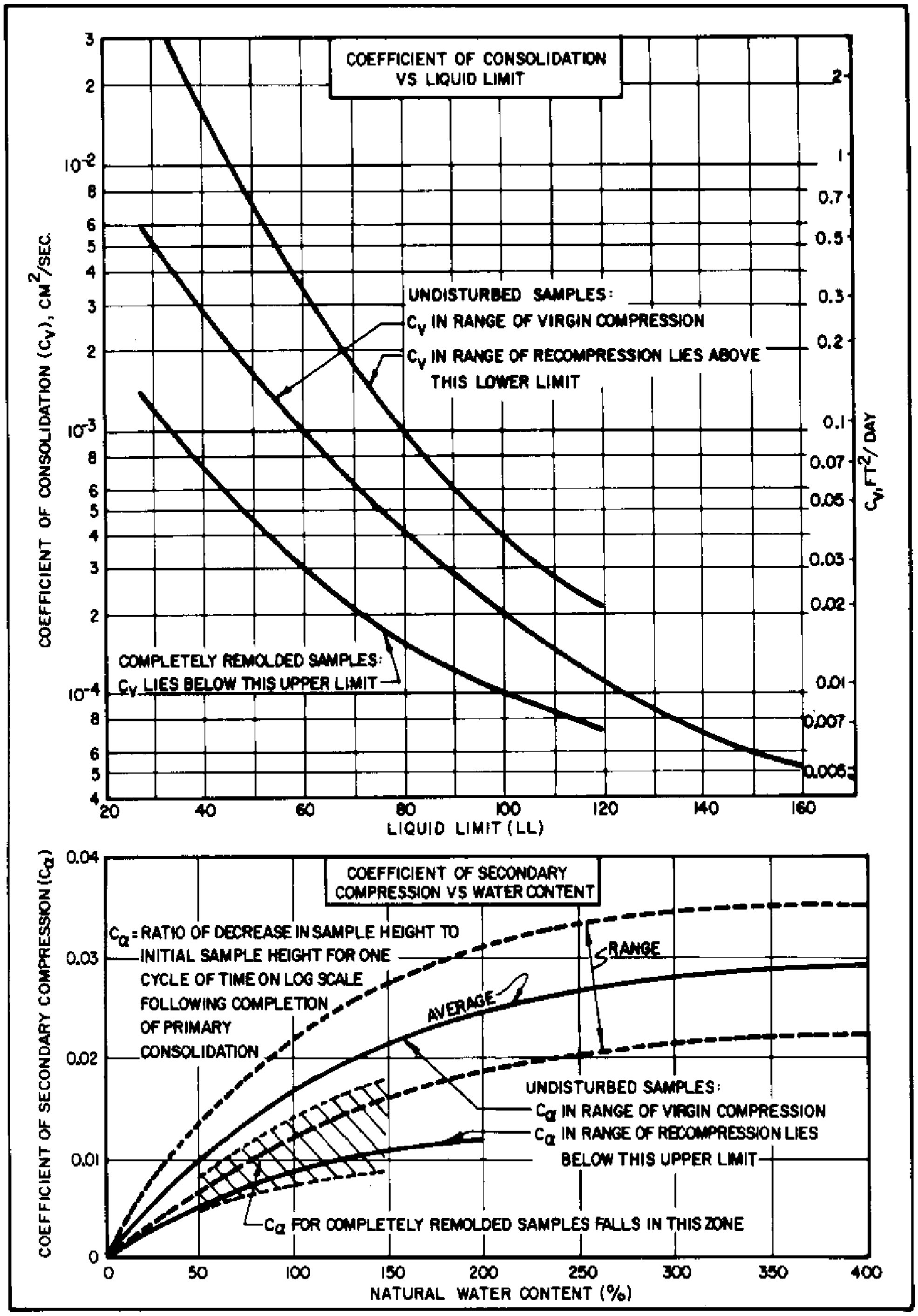


FIGURE 4
Approximate Correlations for Consolidation Characteristics
of Silts and Clays

e. Coefficient of Secondary Compression. Sample disturbance tends to decrease the coefficient of secondary compression in virgin compression loading range.

Section 5. SHEAR STRENGTH TESTS

1. UTILIZATION. The shear strength of soil is required for the analysis of all foundation and earthwork stability problems. Shear strength can be determined by laboratory and field tests, and by approximate correlations with grain size, water content, density, and penetration resistance.

2. TYPES OF SHEAR TESTS. Many types and variations of shear tests have been developed. In most of these tests the rate of deformation is controlled and the resulting loads are measured. In some tests total stress parameters are determined, while in others effective stress strength parameters are obtained. See Chapter 4 for a discussion of total and effective stress concepts. The following are the most widely used testing procedures:

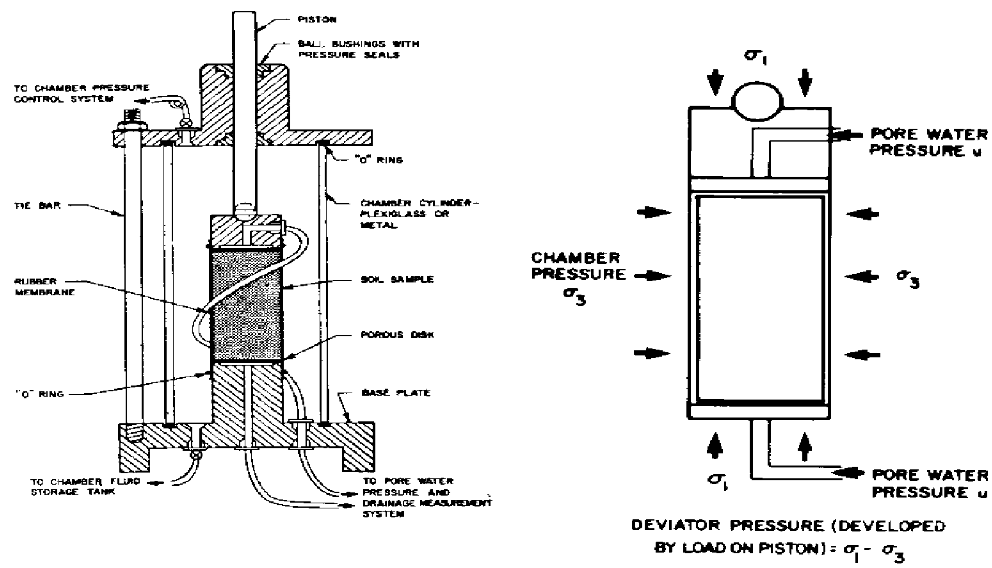
a. Direct Shear Test. A thin soil sample is placed in a shear box consisting of two parallel blocks. The lower block is fixed while the upper block is moved parallel to it in a horizontal direction. The soil fails by shearing along a plane assumed to be horizontal.

This test is relatively easy to perform. Consolidated-drained tests can be performed on soils of low permeability in a short period of time as compared to the triaxial test. However, the stress, strain, and drainage conditions during shear are not as accurately understood or controlled as in the triaxial test.

b. Unconfined Compression Test. A cylindrical sample is loaded in compression. Generally failure occurs along diagonal planes where the greatest ratio of shear stress to shear strength occurs. Very soft material may not show diagonal planes of failure but generally is assumed to have failed when the axial strain has reached a value of 20 percent. The unconfined compression test is performed only on cohesive soil samples. The cohesion (c) is taken as one-half the unconfined compressive strength.

c. Triaxial Compression Test. A cylindrical sample is confined by a membrane and lateral pressure is applied; pore water drainage is controlled through tubing connected to porous discs at the ends of the sample. The triaxial test (Figure 5) permits testing under a variety of loading and drainage conditions and also allows measurement of pore water pressure. For details on testing procedures, see Reference 2. Triaxial shear test relationships are shown graphically in Figure 6.

(1) Unconsolidated-Undrained (UU) or Quick Test (Q). In the UU test the initial water content of the test specimen is not permitted to change during shearing of the specimen.



	UNCONSOLIDATED UNDRAINED TEST	CONSOLIDATED TESTS		
		CONSOLIDATION PHASE	CU	CD
σ_3	HELD CONSTANT	HELD CONSTANT	HELD CONSTANT	HELD CONSTANT
σ_1	GRADUALLY INCREASED FROM σ_3	EQUAL TO σ_3 *	GRADUALLY INCREASED FROM σ_3	VERY GRADUALLY INCREASED FROM σ_3
u	DRAINAGE LINES CLOSED	DRAINAGE LINES OPEN **	NO WATER PERMITTED TO ESCAPE. PORE PRESSURE MEASURED FOR EFFECTIVE STRESS TESTS.	DRAINAGE LINES OPEN

* UNLESS ANISOTROPIC CONSOLIDATION IS TO BE EFFECTED

** IN BACK PRESSURED TESTS, PRESSURE IS SUPPLIED TO PORE LINES, BUT DRAINAGE IS PERMITTED

FIGURE 5
Triaxial Apparatus Schematic

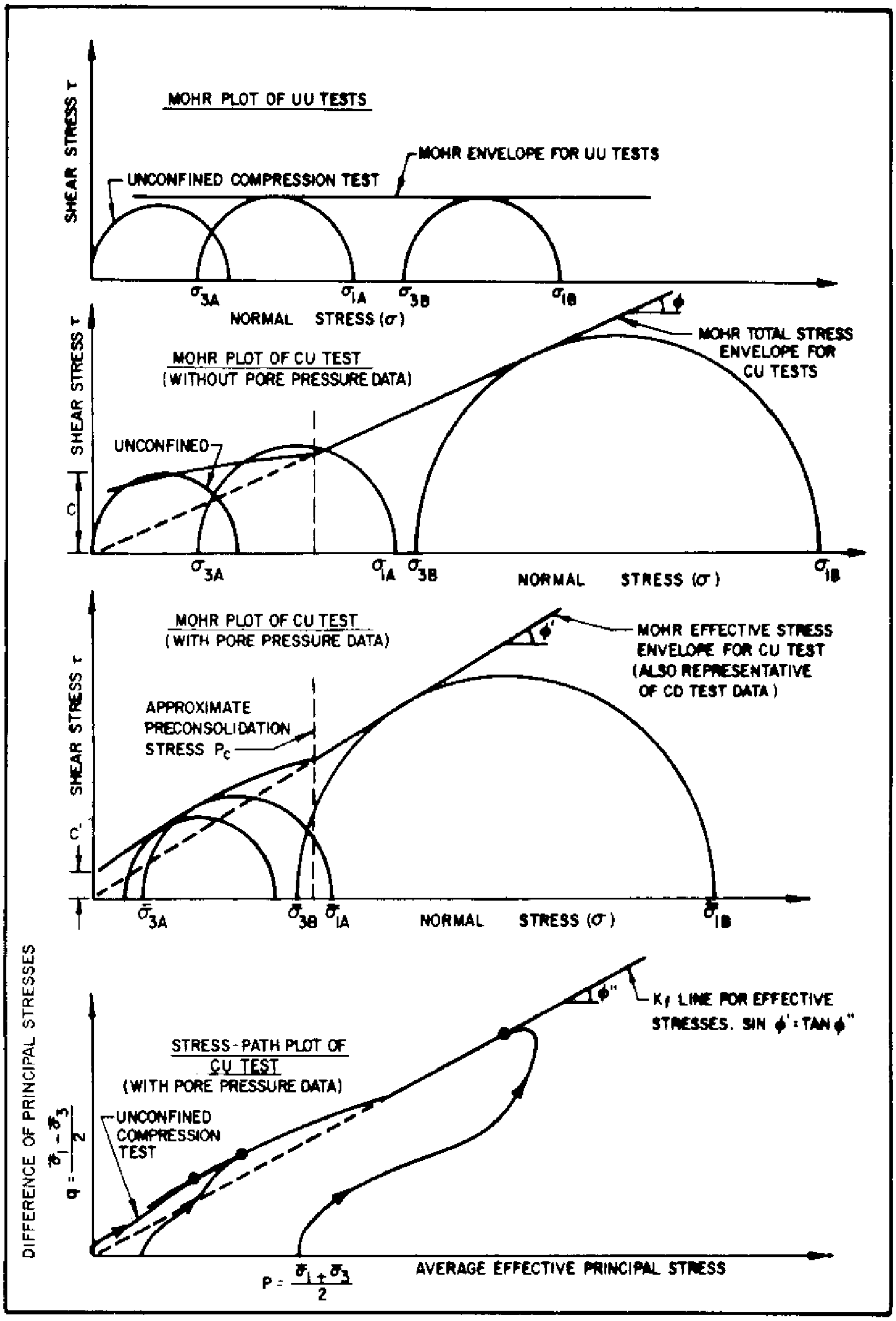


FIGURE 6
Triaxial Shear Test Relationships

The shear strength of soil as determined in UU tests corresponds to total stress, and is applicable only to situations where little consolidation or drainage can occur during shearing. It is applicable primarily to soils having a permeability less than 10. -3- cm per sec.

(2) Consolidated-Undrained (CU) or R Test. In the CU test, complete consolidation of the test specimen is permitted under the confining pressure, but no drainage is permitted during shear. A minimum of three tests is required to define strength parameters c and [phi], though four test specimens are preferable with one serving as a check. Specimens must as a general rule be completely saturated before application of the deviator stress. Full saturation is achieved by back pressure. Pore water pressure is measured during the CU test, thus permitting determination of the effective stress parameters c' and [phi]'. In the absence of pore pressure measurements CU tests can provide only total stress values c and [phi].

(3) Consolidated-Drained (CD) or S Test. In the CD test, complete consolidation of the test speciman is permitted under the confining pressure and drainage is permitted during shear. The rate of strain is controlled to prevent the build-up of pore pressure in the specimen. A minimum of three tests are required for c' and [phi]' determination. CD tests are generally performed on well draining soils. For slow draining soils, several weeks may be required to perform a CD test.

(4) Factors Affecting Tests. Triaxial test results must be appropriately corrected for membrane stiffness, piston friction, and filter drains, whenever applicable. The shear strength of soft sensitive soils is greatly affected by sample disturbance. The laboratory-measured shear strength of disturbed samples will be lower than the in-place strength in the case of UU tests. In the case of CU or CD tests, the strength may be higher because of the consolidation permitted.

d. <u>Other Procedures.</u> In certain instances, more sophisticated tests are warranted. These may include triaxials with zero lateral strain conditions, simple shear tests, and tests inducing anisotropic stress conditions.

3. TEST SELECTION. In determining the type of test to be employed, considerations must be given to soil type and the applications for which the test data is required. (See Chapter 4 for a discussion of total and effective stress concepts.)

a. <u>Soil Type</u>.

(1) Clean Sands and Gravels. Undisturbed samples are very difficult to obtain and test properly, therefore sophisticated shear tests are usually impractical. For simple foundation problems, the angle of internal friction can be satisfactorily approximated by correlation with penetration resistance, relative density, and soil classification (Figure 7). Confirmation of the potential range of the angle of internal friction can be obtained from shear tests on the sample at laboratory densities bracketing conditions anticipated in the field. For earth dam and high embankment work where the soil will be placed under controlled conditions, triaxial compression tests are warranted.

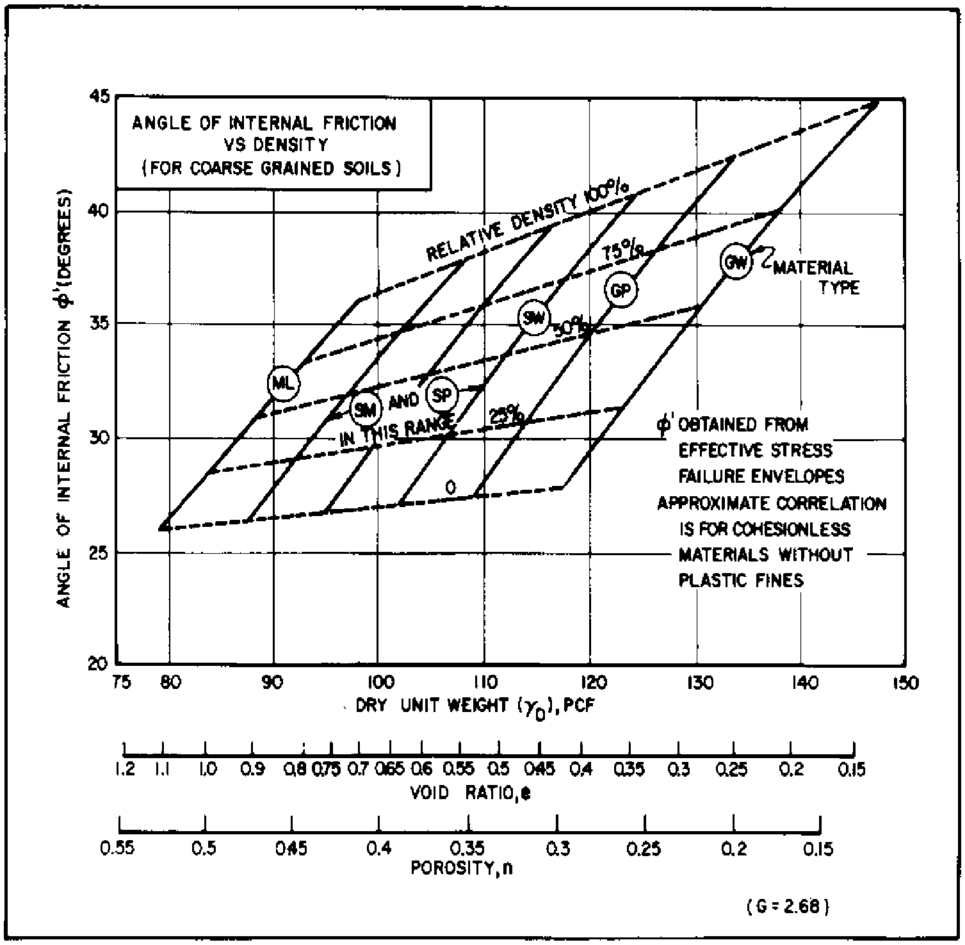


FIGURE 7
Correlations of Strength Characteristics for Granular Soils

(2) Clays. For simple total stress applications where the immediate stability of foundations or embankments is of concern, the unconfined compression test or UU triaxial test is often adequate (Chapter 1). For very soft or sensitive soils, difficult to sample, the field vane test (Chapter 2) is useful. For long-term stability problems requiring effective stress analysis, such as landslides, CU triaxial tests with pore pressure measurements should be used. Long-term stability problems in some highly overconsolidated clays may require the CD test (see Reference 19, Stability of Natural Slopes and Embankment Foundations State-of-the-Art Report, by Skempton and Hutchinson).

(3) Silts and Mixed Soils. The choice of test is governed by whether total stress analysis or effective stress analysis is applicable. In cases of very soft silts, such as in marine deposits, the in-place vane shear test is especially helpful in evaluating the shear strength and its increase with depth. For some thinly layered soils, such as varved clay, direct shear tests or simple shear tests are well suited for determining the strength of the individual layers. Where partial drainage is anticipated, use CU tests with pore water pressure measurements to obtain effective strength parameters.

(4) Overconsolidated Soils. Frequently overconsolidated soils have defects such as jointing, fissures, etc. The laboratory values of strength which are obtained from a small test specimen are generally higher than the field strength values which are representative of the entire soil mass.

The release of stress due to excavation and exposure to weathering reduces strength over a long period of time. This effect cannot be assessed by any of the laboratory tests currently in use. Most overconsolidated clays are anisotropic and the degree of anisotropy may also be influenced by their age. Effect of anisotropy can be determined in the laboratory.

In highly overconsolidated soil which may not be fully saturated, unusually high back pressure may be necessary to achieve full saturation, thus making it difficult to perform CU tests. CD tests are more appropriate.

b. Type of Application.

(1) Total Stress Analysis. It is appropriate for the immediate (during and end of construction) safety of foundations and structures (embankments) consisting of or resting on clays where permeability is low. It is also applicable to embankment stability where rapid drawdown can occur. Use of unconfined compression tests or UU test is appropriate. Sample disturbance has significant effect on shear strength in these types of tests.

(2) Effective Stress Analysis. Evaluation of long-term stability of slopes, embankments, and earth supporting structures in cohesive soil requires the use of effective stress strength parameters, and therefore CU tests with pore water pressure measurements or CD tests are appropriate. Tests must be run at a slow enough strain rate so that pore pressures are equalized during the CU test or are dissipated throughout the CD test. Essentially all analyses of granular soils are made using effective stress.

(3) Stress Path Method. The stress path method is based on modelling the geological and historical stress conditions as they are known to influence soil behavior. To apply the method, stress history is determined and future stresses are computed based on actual construction plans. The stresses are modelled in a set of triaxial or similar strength tests (see Figure 6). Details of this procedure are found in Reference 20, Stress Path Method, Second Edition, by Lambe and Marr.

Section 6. DYNAMIC TESTING

1. UTILIZATION. Capabilities of dynamic soil testing methods and their suitability for various motion characteristics are shown in Table 7 (from Reference 10). Dynamic testing is needed for loose granular soils and soft sensitive clays in earthquake areas, for machine foundation design, and for impact loadings. Only a brief description of tests follows. For further guidance on testing procedures, see References 10 and 11.

2. RESONANT COLUMN TEST. The resonant column test consists of the application of sinusoidal vibration to one end (termed the active end) of a solid or hollow cylindrical soil specimen. The other end is known as the passive end. Compression waves or shear waves are propagated through the soil specimen to determine either Young's modulus (E+s,) or shear modulus (G). Moduli are computed from the resonant frequency of the cylinder. For example, in the case where passive end platen is fixed, the lowest frequency for which the excitation force is in phase with the velocity at the active end is termed the resonant frequency. Damping is determined by turning off the excitation at resonant frequency and recording the decaying vibration.

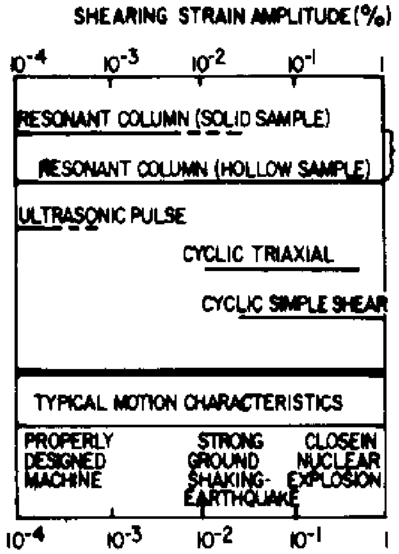
3. CYCLIC TESTS. Currently, these are the most commonly used methods of evaluating the Young's modulus, shear modulus, damping, and liquefaction potential of coarse-grained soils.

a. Cyclic Triaxial Compression Test. In triaxial testing of saturated soils, cell pressure is maintained constant while the axial stress is varied.

b. Cyclic Simple Shear Test. Simple shear equipment has also found wide use in cyclic testing. The non-uniform stress conditions in simple shear may cause failure at a lower stress than that which would cause failure in situ. Measurement or control of lateral pressure is difficult in simple shear tests.

c. Cyclic Torsional Shear. Cyclic torsional simple shear tests on hollow samples offer the capability of measuring lateral confining pressure. In hollow cylinders stresses within the specimen are more uniform, though the specimens are difficult to produce. Also, tapered hollow cyclinders have been used in torsional cyclic tests.

TABLE 7
Capabilities of Dynamic Testing Apparatus



	SHEAR MODULUS G	YOUNGS MODULUS E	DAMPING	CYCLIC STRESS BEHAVIOR	ATTENUATION
RESONANT COLUMN (SOLID SAMPLE) / RESONANT COLUMN (HOLLOW SAMPLE)	X	X	X		
ULTRASONIC PULSE	X	X			X
CYCLIC TRIAXIAL		X	X	X	
CYCLIC SIMPLE SHEAR	X		X	X	

X INDICATE THE PROPERTIES THAT CAN BE DETERMINED.

d. Factors Affecting Tests. Various testing and material factors that may affect cyclic strength as determined in the laboratory are method of specimen preparation, difference between reconstituted and intact specimens, prestressing, loading wave form, grain size and gradation, etc. For details on cyclic testing, see Reference 21, A Review of Factors Affecting Cyclic Triaxial Tests, by Townsend. For the nature of soil behavior under various types of dynamic testing see Reference 22, The Nature of Stress-Strain Behavior for Soils, by Hardin.

4. EMPIRICAL INDICATORS. The empirical relationships given here are to be used only as indicators and not in final design. Design involving dynamic properties of soil must be done only under the direction of experienced personnel.

a. Shear Modulus. In the absence of dynamic tests initial estimates of shear modulus, G, may be made using the relationships found in Reference 23, Shear Modulus and Damping in Soils: Design Equations and Curves, by Hardin and Drnevich, and Reference 24, Soil Moduli and Damping Factors for Dynamic Response Analyses, by Seed and Idriss.

b. Poisson's Ratio. Values of Poisson's ratio ([upsilon]) are generally difficult to establish accurately. For most projects, the value does not affect the response of the structure sufficiently to warrant a great deal of effort in their determination. For cohesionless soils, [upsilon] = 0.25 and for cohesive soils [upsilon] = 0.33 are considered reasonable assumptions. See Reference 25, Foundation Vibration, by Richart.

c. Liquefaction of Coarse-Grained Soils. Liquefaction has usually occurred in relatively uniform material with D+10, ranging between 0.01 and 0.25 mm, C+u, between 2 and 10, and standard penetration resistance less than 25 blows per foot. Liquefaction is more likely to be triggered by higher velocity than by higher acceleration. These characteristics may be used as a guide in determining the need for dynamic testing. The potential influence of local soil conditions (depth of stratum, depth of groundwater table, variation in soil density, etc.) on shaking and damage intensity must be carefully evaluated. See References 26, Earthquake Effects on Soil Foundation Systems, by Seed, and Reference 27, A Practical Method for Assessing Soil Liquefaction Potential Based on Case Studies at Various Sites in Japan, by Iwasaki, et al. A surcharge reduces the tendency of a deposit to liquefy.

Section 7. TESTS ON COMPACTED SOILS

1. UTILIZATION. Compaction is used to densify soils during placement to minimize post-construction consolidation and to improve strength characteristics. Compaction characteristics are determined by moisture density testing; structural and supporting capabilities are evaluated by appropriate tests on samples of compacted soil.

2. MOISTURE-DENSITY RELATIONSHIPS. The Proctor test or a variation is employed in determining the moisture-density relationship. For cohesionless soils, Relative Density methods may be more appropriate.

a. Standard Proctor Test. Use standard Proctor tests for ordinary embankment compaction control. In preparing for control, obtain a family of compaction curves representing principal borrow materials.

b. Modified Proctor Test. Specially applicable to either a heavily compacted base course or a subgrade for airfield pavement and may also be used for mass earthwork.

c. Relative Density of Cohesionless Soils. Proctor tests are often difficult to control for free-draining cohesionless soils and may give erratic compaction curves or density substantially less than those provided by ordinary compaction in the field (see Reference 28, Soil Mechanics, by Lambe and Whitman). Thus, relative density methods may be preferred. Tests for maximum and minimum densities should be done in accordance with ASTM Standard D2049, Relative Density of Cohesionless Soils (Table 3).

3. STRUCTURAL PROPERTIES. Structural properties of compacted-fill materials classified in the Unified System are listed in DM-7.2, Chapter 2, Table 1.

4. CALIFORNIA BEARING RATIO (CBR). This test procedure covers the evaluation of subgrade, subbase, and base course materials for pavement design for highways and airfields. The resistance of a compacted soil to the gradual penetration of a cylindrical piston with 3 square inches in area is measured. The load required to cause either 0.1 inch or 0.2 inch penetration of the piston is compared to that established for a standard compacted crushed stone to obtain the bearing ratio. (See DM-21.03 for approximate relationships between soil type and CBR.) For guidance for design of subbase and bases, see DM-5.04 and DM-21.03.

Section 8. TESTS ON ROCK

1. STRUCTURAL TESTS. Standard methods of testing rock in the laboratory for structural characteristics are only for intact rock. See Table 8 for testing procedures. Behavior of in situ rock, which typically has bedding planes, joints, etc., and may contain discontinuities filled with weaker material, is found to be very different from that of intact rock. In situ tests of joint strengths and compressibility are, therefore, more appropriate. See Chapters 1 and 2 for rock and rock joint classifications and in situ measurements of their properties. The use of data from laboratory tests for bearing and settlement calculations of shallow and deep foundations is shown in DM-7.02 Chapters 4 and 5. Factors which correlate intact rock sample parameters to realistic field parameters are RQD (Rock Quality Designation) or the ratios of field values to laboratory values of compression or shear wave velocities (see Chapters 1 and 2).

TABLE 8
Test Procedures for Intact Rock

Test	Reference for Standard Procedure[(a)]	Size of Sample for Test
Unconfined compressive strength of core specimen	(1, ASTM D2938)	Right circular cylinder with length to diameter ratio of 2 to 2.5, and a diameter not less than 2 inches.
Elastic constants of core specimen	(1, ASTM D3148)	Right circular cylinder with length to diameter ratio of 2 to 2.5.
Direct tensile strength of intact rock core specimen	(1, ASTM D2936)	Right circular cylinder with length to diameter ratio of 2 to 2.5.
Triaxial strength of core specimen	(1, ASTM D2664)	Right circular cylinder with length to diameter ratio of 2 to 2.5.
Dynamic properties of core specimen at small strains	(1, ASTM D2845)	Variable, dependent on properties of specimen and test apparatus.

[(a)] Number in parenthesis indicates Reference number.

2. ROCK QUALITY TESTS.

a. Standards. Quality is normally evaluated by visual examination of the state of weathering and number and condition of discontinuities. RQD provides the best currently available basis for establishing overall rock quality. See Chapter 1 for additional guidance regarding the evaluation of rock quality using RQD. Relative measurements of rock quality can be made by comparing ratios of field values of compression or shear wave velocities to laboratory values (see Chapters 1 and 2).

b. Aggregate Tests. While intended for roadway construction and asphalt and concrete aggregates, there are several standard tests which provide methods for measuring certain aspects of rock quality (see Table 9).

TABLE 9
Test Procedures for Aggregate

Test	Reference for Standard Procedure[(a)]	Applicability to Rock Cores
Weathering resistance.	(1, ASTM C88)	Applicable in principle, can be used directly by fracturing core.
Visual evaluation of rock quality.	(1, ASTM C295)	Direct.
Resistance to freezing.	(1, ASTM C666)	Applicable in principle; but only with significant procedure changes.
Hardness.	(1, ASTM C851)	Direct.

(a)] Number in parenthesis indicates Reference number.

REFERENCES

1. American Society for Testing and Materials, Annual Book of ASTM Standards, Part 19 - Natural Building Stone, Soil and Rock, Peat, Mosses, and Humus; Part 14 - Concrete and Mineral Aggregates; Part 4 -Structural Steel, ASTM, Philadelphia, Pennsylvania.

2. Lambe, T.W., Soil Testing for Engineers, John Wiley, New York, 1951.

3. Bishop, A.W., and Henkel, D.J. The Measurement of Soil Properties in the Triaxial Test, Edward Arnold, Ltd, London, 1962.

4. Office of the Chief of Engineers, Laboratory Soils Testing, Department of the Army, Engineering Manual EM 1110-2-1906, Washington, D.C., 1970.

5. American Society of Agronomy and the American Society for Testing and Materials, Methods of Soil Analysis, Chemical and Microbiological Properties, Part 2, Black, C.A., ed., American Society of Agronomy, Inc., Madison, WI, 1965.

6. National Bureau of Standards, Underground Corrosion, Circular C450, United States Government Printing Office.

7. American Association of State Highway and Transportation Officials, Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Part II, AASHTO, Washington, D.C., 1978.

8. Jennings, J.E. and Knight, K., A Guide to Construction on or with Materials Exhibiting Additional Settlement Due to Collapse of Grain Structures, Sixth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, 1975.

9. Silver, Marshal L., Laboratory Triaxial Testing Procedures to Determine the Cyclic Strength of Soils, prepared under contract to US NRC, (contract No. WRC-E(11-1)-2433), Report No. NUREG-31, 1976.

10. Wood, Richard D., Measurement of Dynamic Soil Properties, ASCE Geotechnical Division Special Conference on Earthquake Engineering and Soil Dynamics, 1978.

11. Drnevich, V.P., Hardin, B.O., and Shippy, D.J., Modulus and Sampling of Soils by the Resonant Column Method, ASTM, STP 654, 1978

12. Stephenson, R.W., Ultrasonic Testing for Determining Dynamic Soil Modulus, ASTM, STP 654, 1978.

13. Bureau of Reclamation, Permeability and Settlement of Soils, Earth Manual, Designation E-13, United States Government Printing Office, 1974.

14. Terzaghi, K. and Peck, R.B., Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, New York, 1967.

15. Wissa, A.E.Z., Christian, J.T., Davis, and Heibert, S., Consolidation at Constant Rate of Strain, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 97, No. SM10, 1971.

16. Lowe, J., New Concepts in Consolidation and Settlement Analysis, Journal of Geotechnical Engineering Division, ASCE, Vol. 101, No. GT6, 1975.

17. Leonards, G.A., Editor, Foundation Engineering, McGraw Hill, 1962.

18. Schmertmann, J.M., The Undisturbed Consolidation of Clay, Transactions, American Society of Civil Engineers, Vol. 120, p 1201, 1955.

19. Skempton, A.W. and Hutchinson, J., Stability of Natural Slopes and Embankment Foundations State-of-the-Art Report, Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969.

20. Lambe, T.W. and Marr, A.W., Stress Path Method, Second Edition, Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT6, 1979.

21. Townsend, F.L., A Review of Factors Affecting Cyclic Triaxial Tests, Dynamic Geotechnical Testing, ASTM, STP 654, pp 356-383, 1978.

22. Hardin, B.O., The Nature of Stress-Strain Behavior for Soils, Earthquake Engineering and Soil Dynamics, Proceedings of the ASCE Geotechnical Engineering Division Specialty Conference, Pasadena, California, pp 3-90, 1978.

23. Hardin, B.O. and Drnevich, V.P., Shear Modulus and Damping in Soils: Design Equations and Curves, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 98, No. SM7, 1972.

24. Seed, H.B. and Idriss, I.M., Soil Moduli and Damping Factors for Dynamic Response Analyses, Report No. EERC 70-10, University of California, 1970.

25. Richart, F.E., Foundation Vibration, Foundation Engineering Handbook, H.F. Winterkown and H.Y. Fang, eds., Van Nostrand Reinhold Company, New York, Chapter 24, 1975.

26. Seed, H.B., Earthquake Effects on Soil Foundation Systems, Foundation Engineering Handbook, H.F. Winterkorn and H.Y. Fang, eds., Van Nostrand Reinhold Company, New York, Chapter 25, 1975.

27. Iwasaki, R., Tatsuoka, F., Tokida, K. and Yasuda, S., A Practical Method for Assessing Soil Liquefaction Potential Based on Case Studies at Various Sites in Japan, Proceedings of the Second International Conference on Microsonation for Safer Construction-Research and Application, Vol. II, San Francisco, pp 885-896, 1978.

28. Lambe, T.W. and Whitman, R.V., Soil Mechanics, John Wiley & Sons, New York, 1969.

29. Naval Facilities Engineering Command, Design Manuals (DM).

DM-5.04	Pavements
DM-21 Series	Airfield Pavement
DM-21.03	Flexible Pavement Design for Airfields

Copies of design manuals may be obtained from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120.

CHAPTER 4. DISTRIBUTION OF STRESSES

Section 1. INTRODUCTION

1. SCOPE. This chapter covers the analysis of stress conditions at a point, stresses beneath structures and embankments, and empirical methods for estimating loads on buried pipes, conduits, shafts, and tunnels.

2. RELATED CRITERIA. For certain criteria not covered in this publication, but concerning the design of buried pipes and conduits and other underground structures, see the following sources:

Subject	Source
Airfield Pavements	NAVFAC DM-21 Series
Drainage Systems	NAVFAC DM-5.03

3. STATE OF STRESS. Stresses in earth masses are analyzed using two basic and different assumptions. One assumes elastic conditions, and the other assumes full mobilization of shear strength (plastic equilibrium). Elastic solutions apply to problems for which shear failure is unlikely. If the safety factor against shear failure exceeds about 3, stresses are roughly equal to values computed from elastic theory. Plastic equilibrium applies in problems of foundation or slope stability (see Chapter 7) and wall pressures where shear strength may be completely mobilized (see DM-7.02, Chapter 3).

Section 2. STRESS CONDITIONS AT A POINT

1. MOHR'S CIRCLE OF STRESS. If normal and shear stresses at one orientation on an element in an earth mass are known, stresses at all other orientations may be determined from Mohr's circle. Examples of stress transformation are given in Figure 1.

a. Plastic Equilibrium. The use of Mohr's circle for plastic equilibrium is illustrated by analysis of triaxial shear test results (see Figure 5 of Chapter 3).

2. STRESSES IN SOILS. The normal stress at any orientation in a saturated soil mass equals the sum of two elements: (a) pore water pressure carried by fluid in soil spaces, and (b) effective stress carried by the grain skeleton of the soil.

a. Total Stress. The total stress at any point is produced by the overburden pressure plus any applied loads.

b. Pore Water Pressure. Pore water pressure may consist of (a) hydrostatic pressure, (b) capillary pressure, (c) seepage or (d) pressure resulting from applied loads to soils which drain slowly.

c. Effective Stress. Effective stress equals the total stress minus the pore water pressure, or the total force in the soil grains divided by the gross cross-sectional area over which the force acts.

d. Overburden Pressure. Division of weight of overlying soil and water into effective stress and pore water pressure depends on the position of the groundwater table or the flow field induced by seepage. For static water condition, effective stresses at any point below the groundwater level may be computed using the total unit weight of soil above the water level and buoyant unit weight below the water level. Pore water pressure is equal to the static head times the unit weight of water. If there is steady seepage, pore pressure is equal to the piezometric head times the unit weight of water, and the effective stress is obtained by subtracting the pore water pressure from the total stress.

e. Applied Load. Division of applied load between pore pressure and effective stress is a function of the boundary conditions, the stress-strain properties, and the permeability of the stressed and surrounding soils. When drainage of pore water is inhibited, load is compensated for by increased pore water pressures. These pressures may decrease with time, as pore water is drained and load is transferred to the soil skeleton, thereby increasing effective stress. Guidance on estimating changes in pore water pressure is given in Chapter 5.

f. Effects of Stresses on a Soil Mass. Analysis of a soil system (e.g., settlement, stability analyses) are performed either in terms of total stresses or effective stresses. The choice between the two analysis methods is governed by the properties of the surrounding soils, pore water behavior, and the method of loading. (See Chapters 5, 6, and 7 for further discussion.)

Section 3. STRESSES BENEATH STRUCTURES AND EMBANKMENTS

1. SEMI-INFINITE, ELASTIC FOUNDATIONS.

a. Assumed Conditions. The following solutions assume elasticity, continuity, static equilibrium, and completely flexible loads so that the pressures on the foundation surface are equal to the applied load intensity. For loads of infinite length or where the length is at least 5 times the width, the stress distribution can be considered plane strain, i.e., deformation occurs only in planes perpendicular to the long axis of the load. In this case stresses depend only on direction and intensity of load and the location of points being investigated and are not affected by elastic properties.

Shearing stresses between an embankment and its foundation are neglected.

b. Stress Distribution Formulas. Figure 2 presents formulas based on the Boussinesq equations for subsurface stresses produced by surface loads on semi-infinite, elastic, isotropic, homogeneous foundations. Below a depth of three times the width of a square footing or the diameter of a circular footing, the stresses can be approximated by considering the footing to be a point load. A strip load may also be treated as a line load at depths greater than three times the width of the strip.

c. Vertical Stresses Beneath Regular Loads. Charts for computations of vertical stress based on the Boussinesq equations are presented in Figures 3 through 7. Use of the influence charts is explained by examples in Figure 8. Computation procedures for common loading situations are as follows:

(1) Square and Strip Foundations. Quick estimates may be obtained from the stress contours of Figure 3. For more accurate computations, use Figure 4 (Reference 1, Stresses and Deflections in Foundations and Pavements, by the Department of Civil Engineering, University of California, Berkeley).

(2) Rectangular Mat Foundation. For points beneath the mat, divide the mat into four rectangles with their common corner above the point to be investigated. Obtain influence values I for the individual rectangles from Figure 4, and sum the values to obtain the total I. For points outside the area covered by the mat, use superposition of rectangles and add or subtract appropriate I values to obtain the resultant I. (See example in Figure 9.)

(3) Uniformly Loaded Circular Area. Use Figure 5 (Reference 2, Stresses and Deflections Induced by Uniform Circular Load, by Foster and Ahlvin) to compute stresses under circular footings.

(4) Embankment of Infinite Length. Use Figure 6 (Reference 3, Influence Values for Vertical Stresses in a Semi-Infinite Mass Due to an Embankment Loading, by Osterberg) for embankments of simple cross section. For fills of more complicated cross section, add or subtract portions of this basic embankment load. For a symmetrical triangular fill, set dimension b equal to zero and add the influence values for two right triangles.

(5) Sloping Fill of Finite Dimension. Use Figure 7 (Reference 1) for stress beneath the corners of a finite sloping fill load.

d. Vertical Stresses Beneath Irregular Loads. Use Figure 10 (Reference 4, Soil Pressure Computations: A Modification of Newmark's Method, by Jimenez Salas) for complex loads where other influence diagrams do not suffice. Proceed as follows:

(1) Draw a circle of convenient scale and the concentric circles shown within it. The scale for the circle may be selected so that when the foundation plan is drawn using a standard scale (say 1"=100'), it will lie within the outer circle.

(2) Plot the loaded area to scale on this target with the point to be investigated at the center.

(3) Estimate the proportion A of the annular area between adjacent radii which is covered by the load.

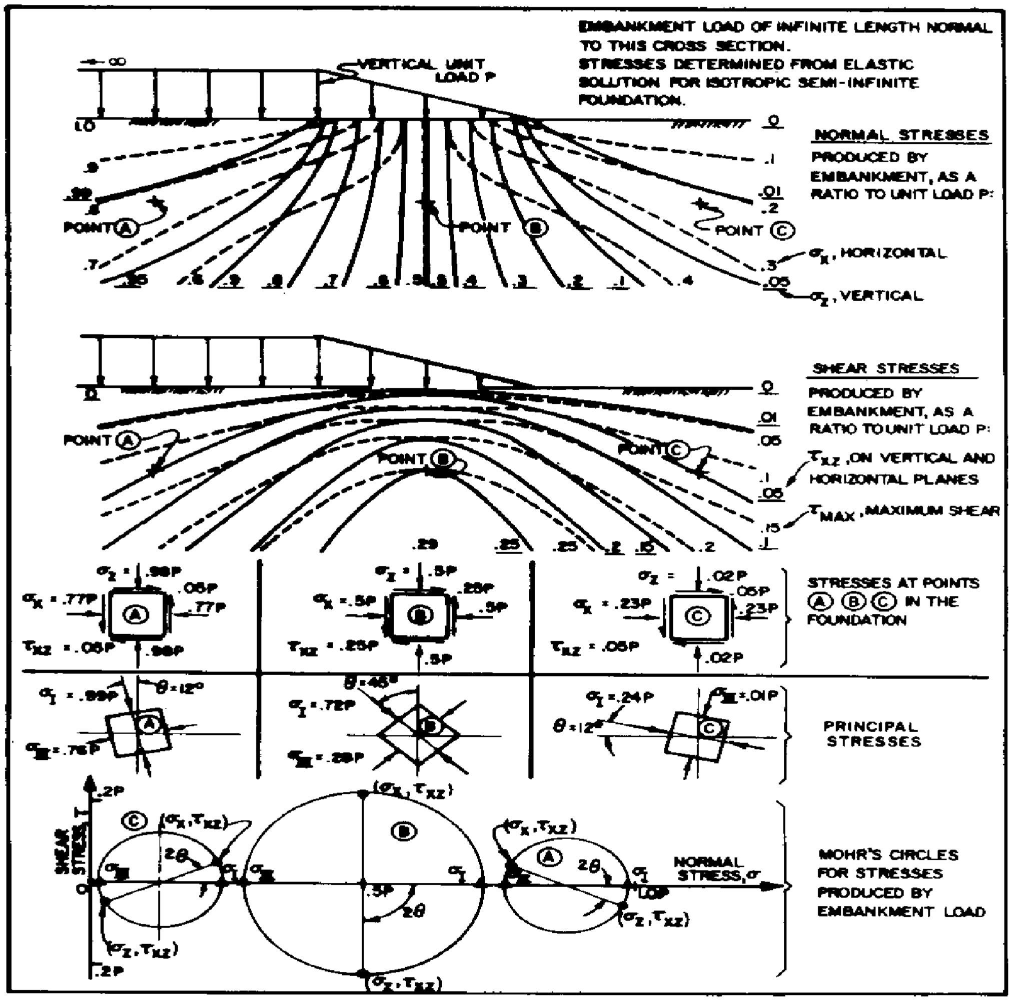
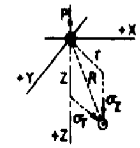
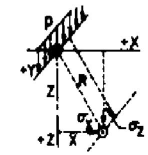
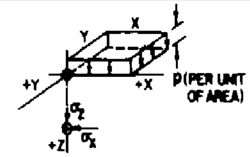
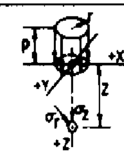
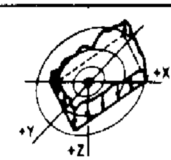
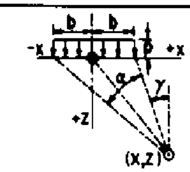
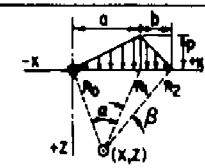
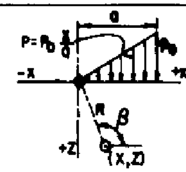
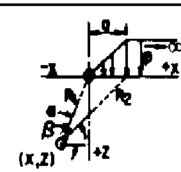
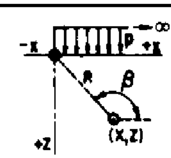


FIGURE 1
Examples of Stress Conditions at a Point

LOADING CONDITION	STRESS DIAGRAM	STRESS COMPONENT	EQUATION
POINT LOAD		VERTICAL	$\sigma_z = -\frac{P}{2\pi R^2}\left[\frac{-3r^2 Z}{R^3} + \frac{(1-2\mu)R}{R+Z}\right]$
		HORIZONTAL	$\sigma_r = \frac{P}{2\pi}\left[3\frac{r^2 Z}{R^5} - (1-2\mu)\left(\frac{R-Z}{Rr^2}\right)\right]$
		SHEAR	$\tau_{rz} = \frac{3P}{2\pi} \cdot \frac{rZ^2}{R^5}$
UNIFORM LINE LOAD OF INFINITE LENGTH		VERTICAL	$\sigma_z = \frac{2p}{\pi} \cdot \frac{Z^3}{R^4}$
		HORIZONTAL	$\sigma_x = \frac{2p}{\pi} \cdot \frac{x^2 Z}{R^4}$
		SHEAR	$\tau_{xz} = \frac{2p}{\pi} \cdot \frac{xZ^2}{R^4}$
UNIFORMLY LOADED RECTANGULAR AREA (FIGURE 4)		VERTICAL (BENEATH CORNER OF RECTANGLE)	$\sigma_z = \frac{p}{4\pi}\left[\frac{2XYZ(X^2+Y^2+Z^2)^{1/2}}{Z^2(X^2+Y^2+Z^2)+X^2Y^2} \cdot \frac{X^2+Y^2+2Z^2}{X^2+Y^2+Z^2} + \tan^{-1}\frac{2XYZ(X^2+Y^2+Z^2)^{1/2}}{Z^2(X^2+Y^2+Z^2)-X^2Y^2}\right]$
UNIFORMLY LOADED CIRCULAR AREA (FIGURE 5)		VERTICAL	$\sigma_z = p\left\{1 - \frac{1}{[1+(r/Z)^2]^{3/2}}\right\}$
		HORIZONTAL	$\sigma_r = \frac{p}{2}\left[1+2\mu-2(1+\mu)\left(\frac{Z}{\sqrt{r^2+Z^2}}\right)+\left(\frac{Z}{\sqrt{r^2+Z^2}}\right)^3\right]$
		SHEAR	$\tau_{rz} = 0$ (STRESS COMPONENTS $\sigma_z, \sigma_r, \tau_{rz}$ BENEATH CENTER OF CIRCLE)
IRREGULAR LOAD		VERTICAL	COMPUTED FROM INFLUENCE CHART OF FIGURE 10

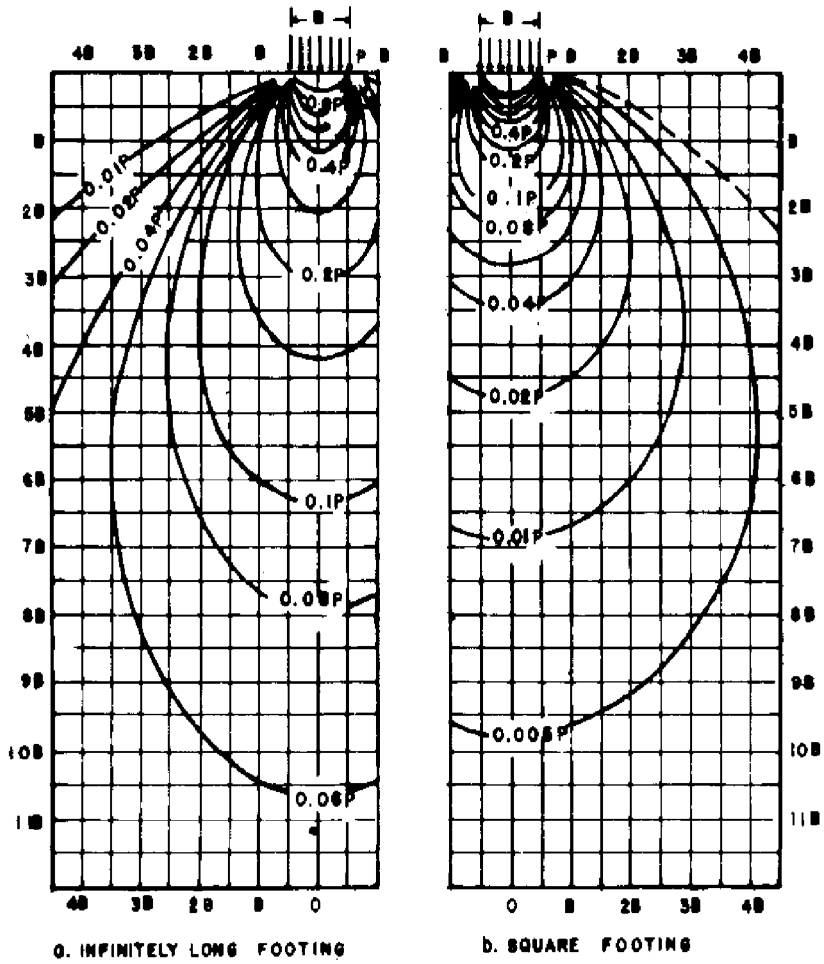
ASSUMED CONDITIONS: APPLIED LOADS ARE PERFECTLY FLEXIBLE. FOUNDATION IS SEMI-INFINITE ELASTIC ISOTROPIC SOLID.

FIGURE 2
Formulas for Stresses in Semi-Infinite Elastic Foundation

	LOADING CONDITION	STRESS DIAGRAM	STRESS COMPONENT	EQUATION
EMBANKMENT LOADS OF INFINITE LENGTH	UNIFORM STRIP LOAD		VERTICAL	$\sigma_z = \frac{p}{\pi}\left[\alpha + \sin\alpha \cdot \cos(\alpha + 2\gamma)\right]$
			HORIZONTAL	$\sigma_x = \frac{p}{\pi}\left[\alpha - \sin\alpha \cdot \cos(\alpha + 2\gamma)\right]$
			SHEAR	$\tau_{xz} = \frac{p}{\pi}\left[\sin\alpha \cdot \sin(\alpha + 2\gamma)\right]$
	TRIANGULAR LOAD		VERTICAL	$\sigma_z = \frac{p}{\pi}\left[\frac{x}{a}\alpha + \frac{a+b-x}{b}\beta\right]$
			HORIZONTAL	$\sigma_x = \frac{p}{\pi}\left[\frac{x}{a}\alpha + \frac{a+b-x}{b}\beta + \frac{2Z}{a}\log_e\frac{R_1}{R_0} + \frac{2Z}{b}\log_e\frac{R_1}{R_2}\right]$
			SHEAR	$\tau_{xz} = \frac{pZ}{\pi}\left[\frac{\alpha}{a} - \frac{\beta}{b}\right]$
	SLOPE LOAD		VERTICAL	$\sigma_z = \frac{p_0}{\pi a}\left[X\beta + Z\right]$
			HORIZONTAL	$\sigma_x = \frac{p_0}{\pi a}\left[X\beta - Z - 2Z\log_e R\right]$
			SHEAR	$\tau_{xz} = \frac{p_0}{\pi a} Z\beta$
	TERRACE LOAD		VERTICAL	$\sigma_z = \frac{p}{\pi a}\left[a\beta + X\alpha\right]$
			HORIZONTAL	$\sigma_x = \frac{p}{\pi a}\left[a\beta + X\alpha + 2Z\log_e\frac{R_2}{R_1}\right]$
			SHEAR	$\tau_{xz} = \frac{p}{\pi a} \cdot Z\alpha$
	SEMI-INFINITE UNIFORM LOAD		VERTICAL	$\sigma_z = \frac{p}{\pi}\left[\beta + \frac{XZ}{R^2}\right]$
			HORIZONTAL	$\sigma_x = \frac{p}{\pi}\left[\beta - \frac{XZ}{R^2}\right]$
			SHEAR	$\tau_{xz} = -\frac{p}{\pi} \cdot \sin^2\beta$

ASSUMED CONDITIONS: APPLIED LOADS ARE PERFECTLY FLEXIBLE. FOUNDATION IS SEMI-INFINITE ELASTIC ISOTROPIC SOLID.

FIGURE 2 (continued)
Formulas for Stresses in Semi-Infinite Elastic Foundation



SQUARE FOOTING

GIVEN

FOOTING SIZE = 20' X 20'

UNIT PRESSURE P = 2 TSF

FIND

PROFILE OF STRESS INCREASE BENEATH CENTER OF FOOTING DUE TO APPLIED LOAD

B = 20' P = 2 TSF

z (FT)	$\frac{z}{B}$	σ_z TSF
10	0.5	0.70 X 2 = 1.4
20	1	0.38 X 2 = 0.76
30	1.5	0.19 X 2 = 0.38
40	2.0	0.12 X 2 = 0.24
50	2.5	0.07 X 2 = 0.14
60	3.0	0.05 X 2 = 0.10

FIGURE 3
Stress Contours and Their Application

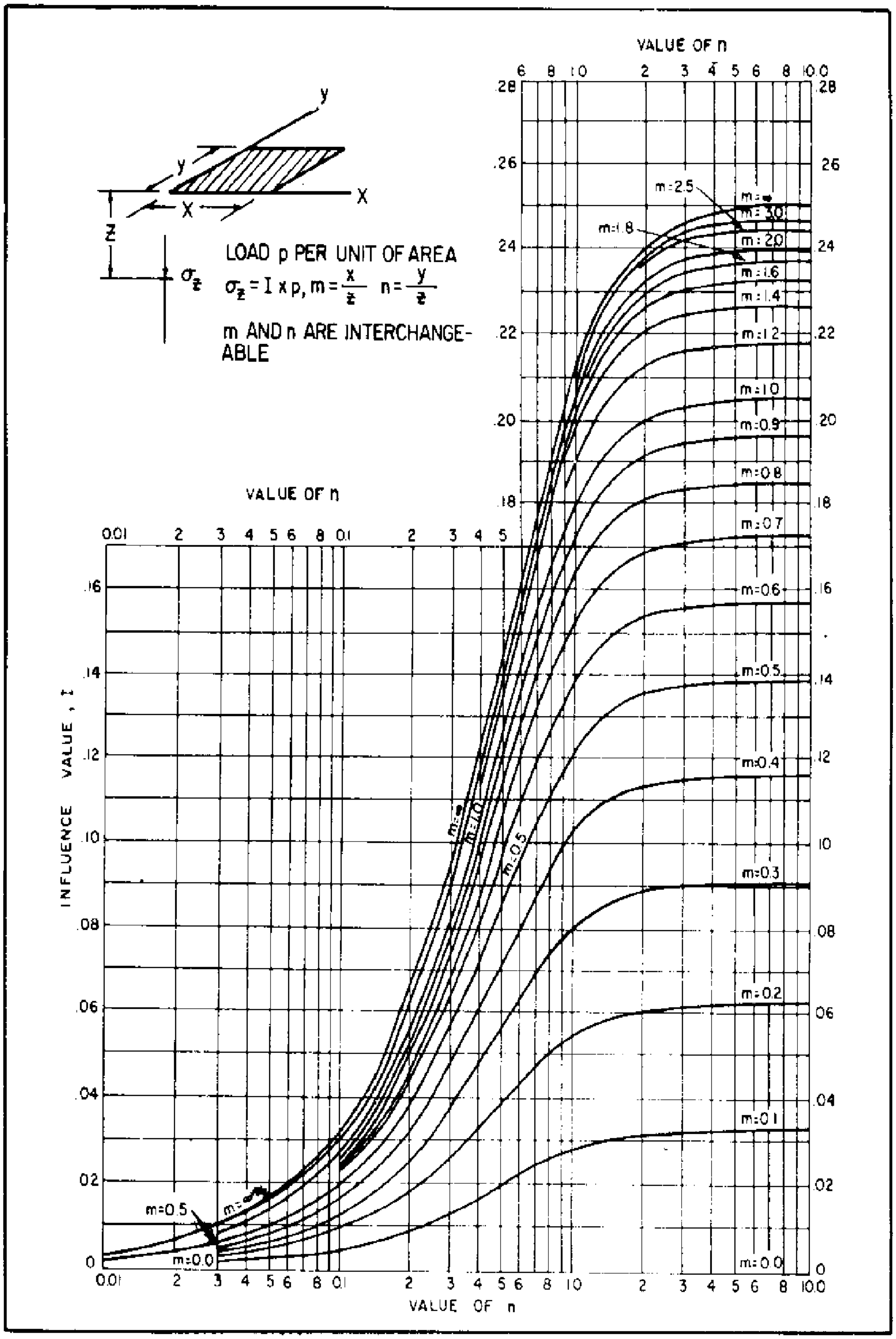


FIGURE 4
Influence Value for Vertical Stress Beneath a Corner of a Uniformly Loaded Rectangular Area (Boussinesq Case)

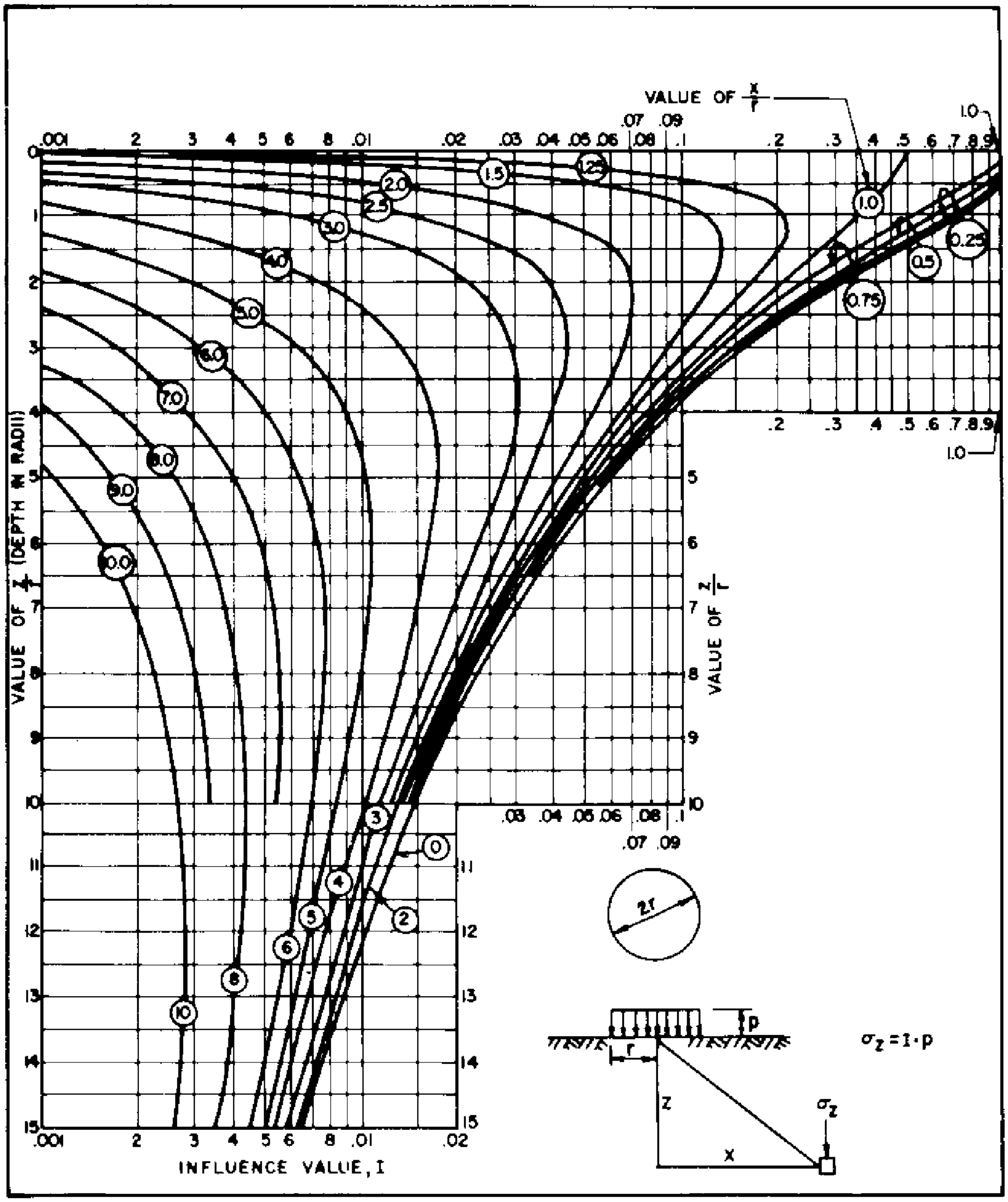


FIGURE 5
Influence Value for Vertical Stress Under Uniformly Loaded Circular Area
(Boussinesq Case)

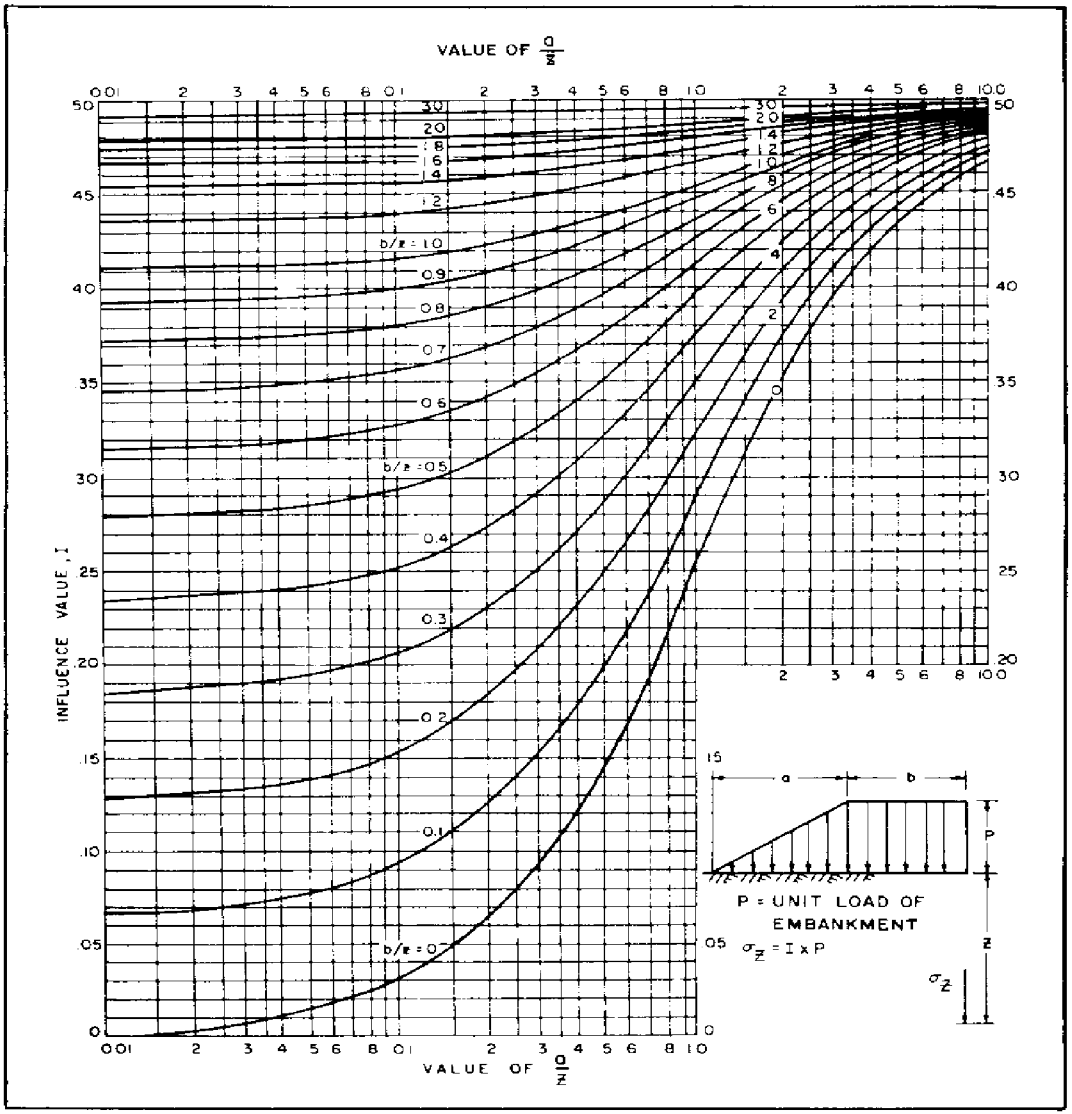


FIGURE 6
Influence Value for Vertical Stress Under Embankment Load of Infinite Length
(Boussinesq Case)

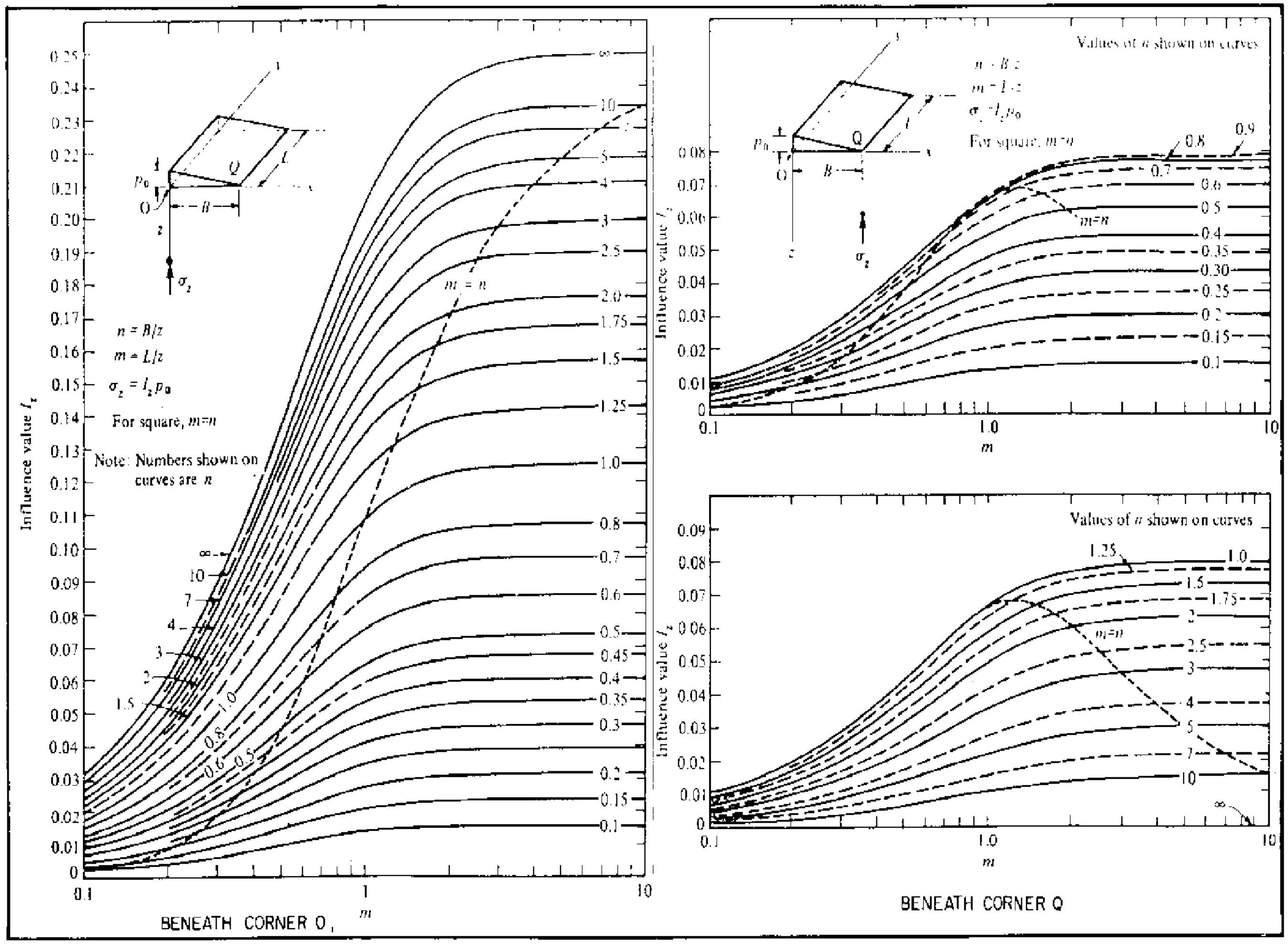
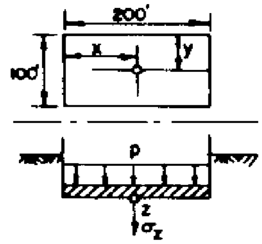


FIGURE 7
Influence Value for Vertical Stress Beneath Triangular Load
(Boussinesq Case)

RECTANGULAR MAT FOUNDATION

REFER TO FIGURE 4 FOR INFLUENCE VALUES I FOR VERTICAL STRESS UNDER CORNER OF UNIFORMLY LOADED RECTANGULAR AREA. I FOR 1 OF 4 ADJOINING RECTANGULAR AREAS.



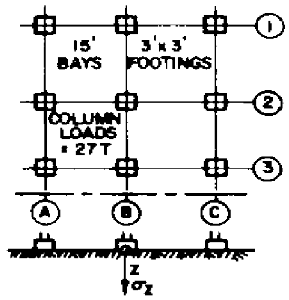
$x = 100'$
$y = 50'$
$p = 1.2$ TSF
$m = \frac{x}{z}$
$n = \frac{y}{z}$

z FT.	m	n	I 1 RECT.	4I 4 RECT.	$\sigma_z = 4Ip$ TSF
5	20	10	0.250	1.000	1.200
10	10	5	0.249	0.996	1.195
20	5	2.5	0.244	0.976	1.171
30	3.33	1.67	0.232	0.928	1.113
40	2.50	1.25	0.216	0.864	1.037
60	1.67	0.83	0.182	0.728	0.874
80	1.25	0.63	0.149	0.596	0.715
100	1.00	0.50	0.126	0.504	0.605

UNIT AREA LOAD (INCL. MAT) = 2 TSF, UNIT LOAD EXCAVATED = .8 TSF
NET AREA LOAD = 1.2 TSF. DETERMINE PROFILE OF APPLIED STRESS BENEATH CENTER OF MAT

SEPARATE COLUMN FOOTINGS

REFER TO FIGURE 5 FOR INFLUENCE VALUES I FOR VERTICAL STRESS OUTSIDE OF UNIFORMLY LOADED CIRCULAR AREA. SQUARE FOOTING ASSUMED EQUIVALENT TO CIRCULAR AREA.



EQUIVALENT r:
$r = \left(\frac{9}{\pi}\right)^{1/2} = 1.695'$
$p = 3.0$ TSF

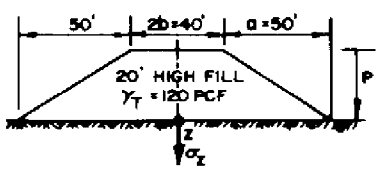
X DISTANCE:
COL. B2 = 0 (I_1)
COLS. A2, C2, B1, B3 = 15' (I_2)
COLS. A1, A3, C1, C3 = 21.2' (I_3)

z FT.	$\frac{z}{r}$	I_1 x/r=0	$I_2/4$ x/r=8.9	$I_3/4$ x/r=12.5	ΣI	$\sigma_z = \Sigma I \cdot p$ TSF
2	1.18	0.64	0	0	0.640	1.720
4	2.36	0.22	0	0	0.220	0.660
6	3.54	0.110	0	0	0.110	0.330
10	5.90	0.042	0.002	0	0.050	0.150
15	8.85	0.019	0.003	0.001	0.035	0.105
20	11.80	0.011	0.004	0.002	0.035	0.105
25	14.74	0.007	0.003	0.002	0.027	0.081

DETERMINE PROFILE OF APPLIED STRESS BENEATH B2.

EMBANKMENT LOAD

REFER TO FIGURE 6 FOR INFLUENCE VALUES I FOR VERTICAL STRESS UNDER EMBANKMENT LOAD OF INFINITE LENGTH. I FOR 1/2 OF EMBANKMENT LOAD.



$a = 50'$
$b = 20'$
$p = 1.2$ TSF

z FT.	$\frac{a}{z}$	$\frac{b}{z}$	I	2I	$\sigma_z = 2I \cdot p$ TSF
5	10	4	0.500	1.000	1.200
10	5	2	0.496	0.992	1.190
20	2.5	1.0	0.475	0.950	1.140
30	1.67	0.67	0.444	0.888	1.066
40	1.25	0.50	0.410	0.820	0.984
60	0.83	0.33	0.344	0.688	0.826
80	0.63	0.25	0.294	0.588	0.706
100	0.50	0.20	0.252	0.504	0.605

DETERMINE PROFILE OF APPLIED STRESS σ_z BENEATH ℄ OF EMBANKMENT OF INFINITE LENGTH.

FIGURE 8
Examples of Computation of Vertical Stress

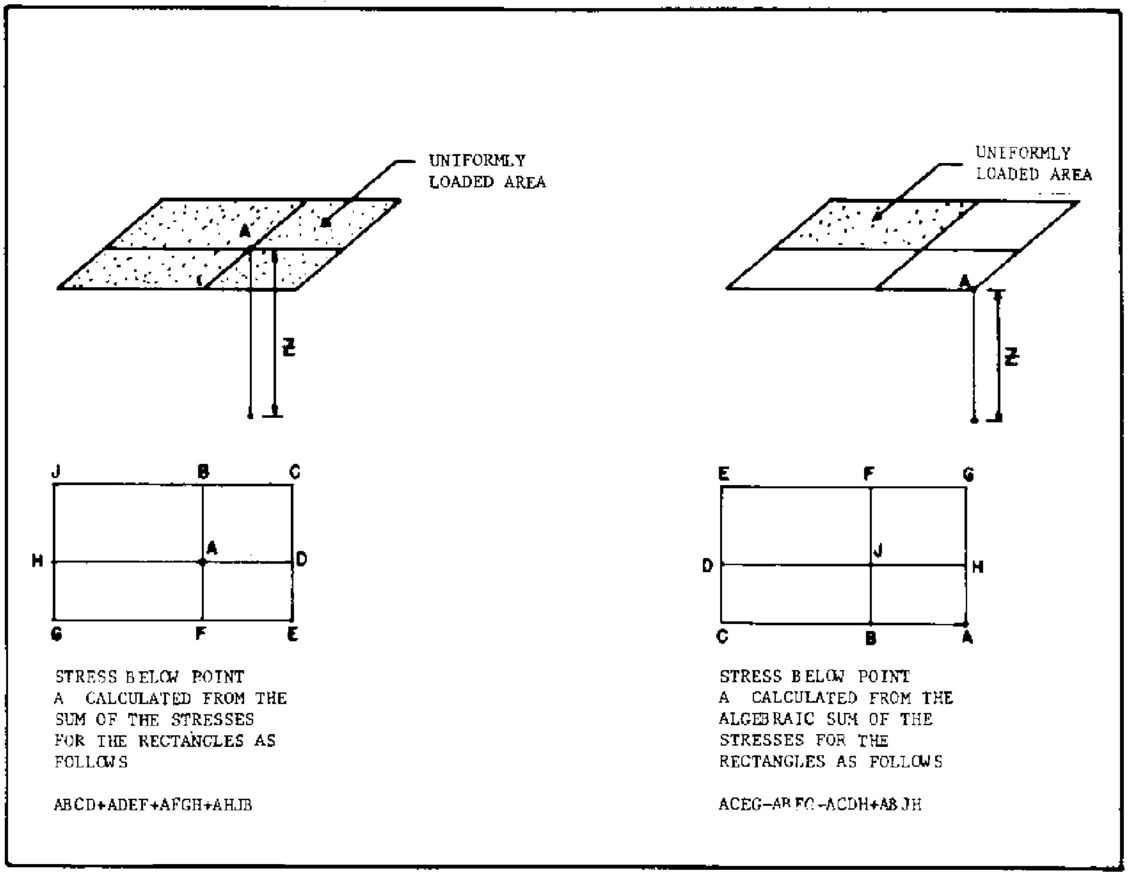
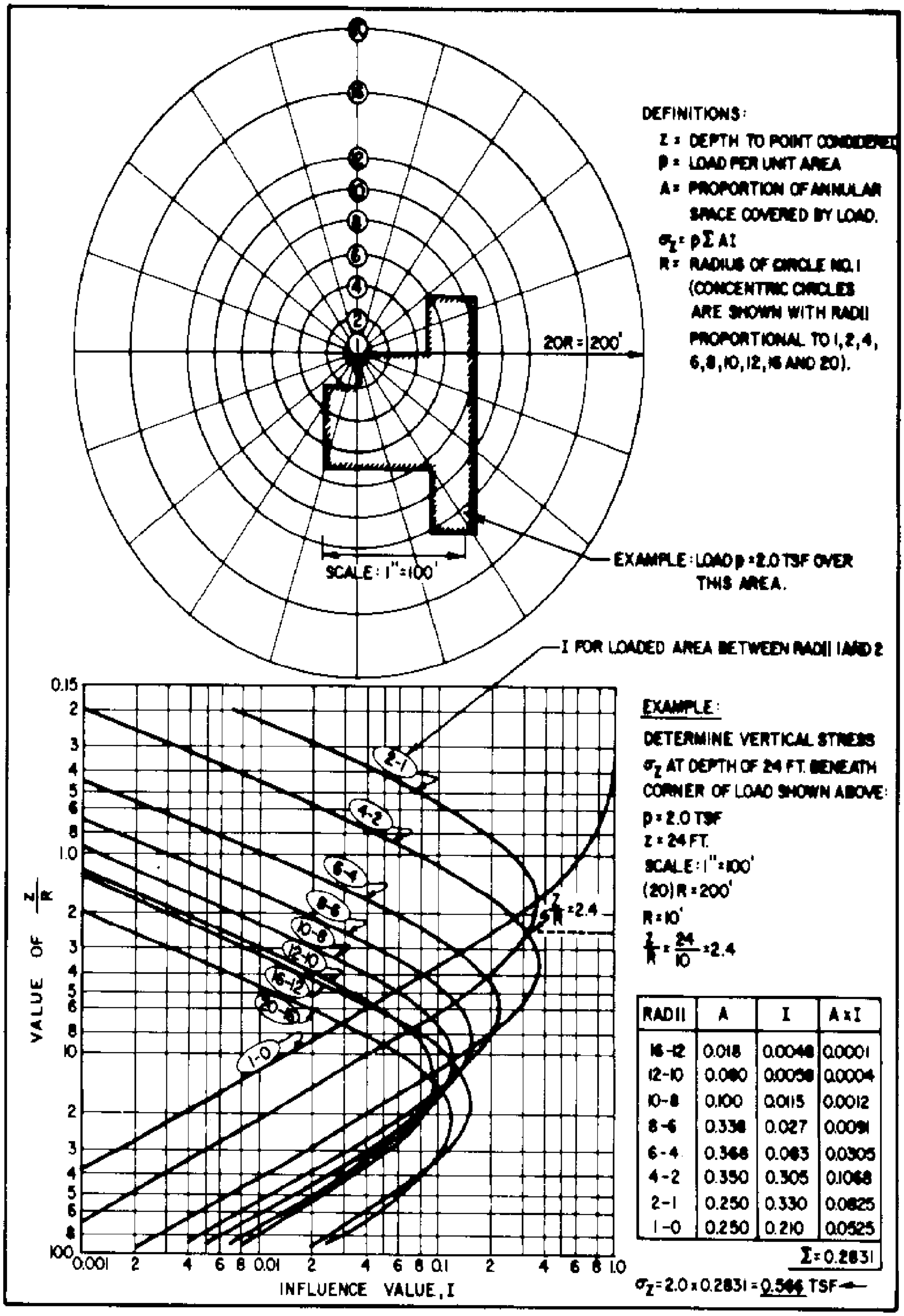


FIGURE 9
Determination of Stress Below Corner of
Uniformly Loaded Rectangular Area



RADII	A	I	A x I
16-12	0.018	0.0048	0.0001
12-10	0.080	0.0058	0.0004
10-8	0.100	0.0115	0.0012
8-6	0.338	0.027	0.0091
6-4	0.368	0.083	0.0305
4-2	0.350	0.305	0.1068
2-1	0.250	0.330	0.0825
1-0	0.250	0.210	0.0525
			$\Sigma = 0.2831$

$\sigma_z = 2.0 \times 0.2831 = \underline{0.566}$ TSF

FIGURE 10
Influence Chart for Vertical Stress Beneath Irregular Load

(4) See the bottom chart of Figure 10 for influence values for stresses at various depths produced by the loads within each annular space. The product I x A multiplied by the load intensity equals vertical stress.

(5) To determine a profile of vertical stresses for various depths beneath a point, the target need not be redrawn. Obtain influence values for different ordinates Z/R from the influence chart.

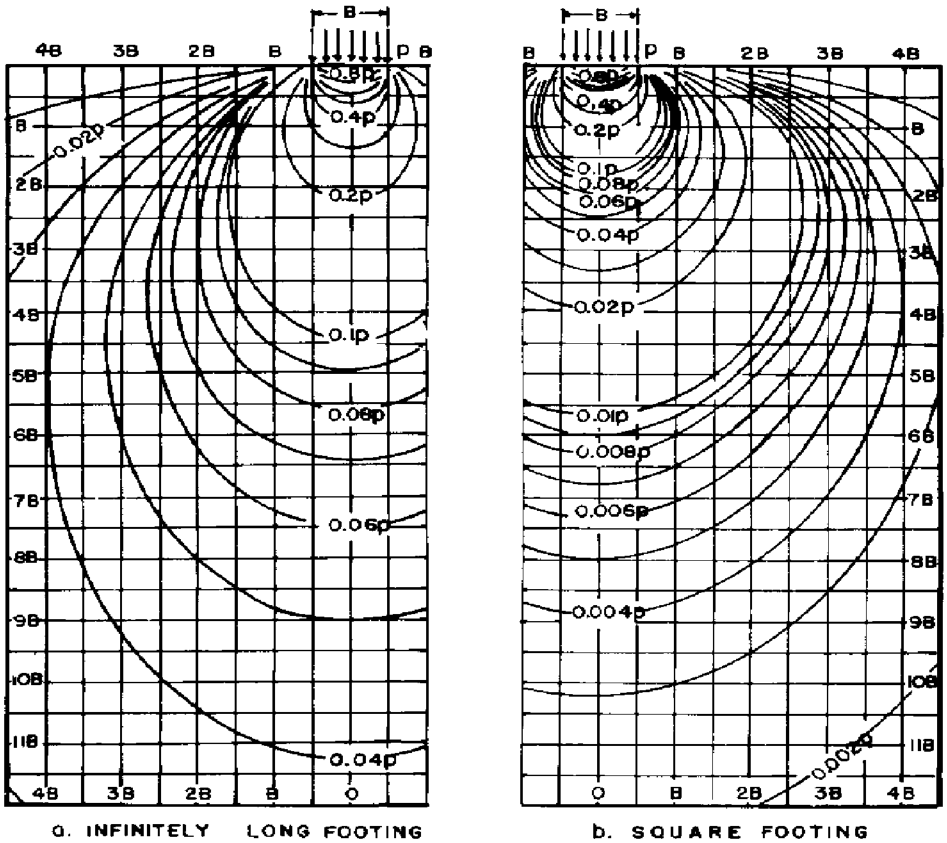
e. Horizontal Stresses. Elastic analysis is utilized to determine horizontal stresses on unyielding walls from surcharge loads (see Chapter 7.02, Chapter 3), and pressures on rigid buried structures. (See basic formulas for simple loads in Figure 2.) For more information, see Reference 5, Elastic Solutions for Soil and Rock Mechanics, by Poulos and Davis.

f. Shear Stresses. Elastic solutions generally are not applicable when shear stresses are critical, as in stability problems. To determine if a stability analysis is required, determine the maximum shear stress from elastic formulas and compare this stress with the shear strength of the soil. For embankment loads in Figure 2, maximum shear stress in the foundation is exactly or approximately equal to p/[pi] depending upon the shape of the load and point in question. If the maximum shear stress equals shear strength, plastic conditions prevail at some point in the foundation soil and if the load is increased, a larger and larger portion of the foundation soil passes into plastic equilibrium. In this case, failure is possible and overall stability must be evaluated.

2. LAYERED OR ANISOTROPIC FOUNDATIONS. Actual foundation conditions differ from the homogeneous isotropic, semi-infinite mass assumed in the Boussinesq expressions. The modulus of elasticity usually varies from layer to layer, and soil deposits frequently are more rigid in the horizontal direction than in the vertical.

a. Westergaard Analysis. The Westergaard analysis is based on the assumption that the soil on which load is applied is reinforced by closely spaced horizontal layers which prevent horizontal displacement. The effect of the Westergaard assumption is to reduce the stresses substantially below those obtained by the Boussinesq equations. The Westergaard analysis is applicable to soil profiles consisting of alternate layers of soft and stiff materials, such as soft clays with frequent horizontal layers of sand having greater stiffness in the horizontal direction. Figures 11 (Reference 1), 12 (Reference 6, An Engineering Manual for Settlement Studies, by Duncan and Buchignani), and 13 (Reference 1) can be used for calculating vertical stresses in Westergaard material for three loading conditions. Computations for Figures 11, 12, and 13 are made in a manner identical to that for Figures 3, 4, and 7, which are based on the Boussinesq equations. For illustration see Figure 8.

b. Layered Foundations. When the foundation soil consists of a number of layers of substantial thickness, having distinctly different elastic properties, the vertical and other stresses are markedly different from those obtained by using the Boussinesq equation. (See Figure 14, Reference 7, Stresses and Displacement in Layered Systems, by Mehta and Veletsos, for influence values of vertical stresses in a two-layer foundation with various ratios of modulus of elasticity. See Figure 15 for an example.)



EXAMPLE:

FIND THE PRESSURE INCREASE DUE TO A STRIP FOOTING OF WIDTH B, AT A POINT LOCATED 6B BELOW ITS BASE AND 3B FROM THE CENTER OF THE FOOTING. SURFACE LOAD ON THE FOOTING IS P PER-UNIT AREA. FROM THE LEFT PANEL, PRESSURE INCREASE = 0.05 P

FIGURE 11
Vertical Stress Contours for Square and Strip Footings
(Westergaard Case)

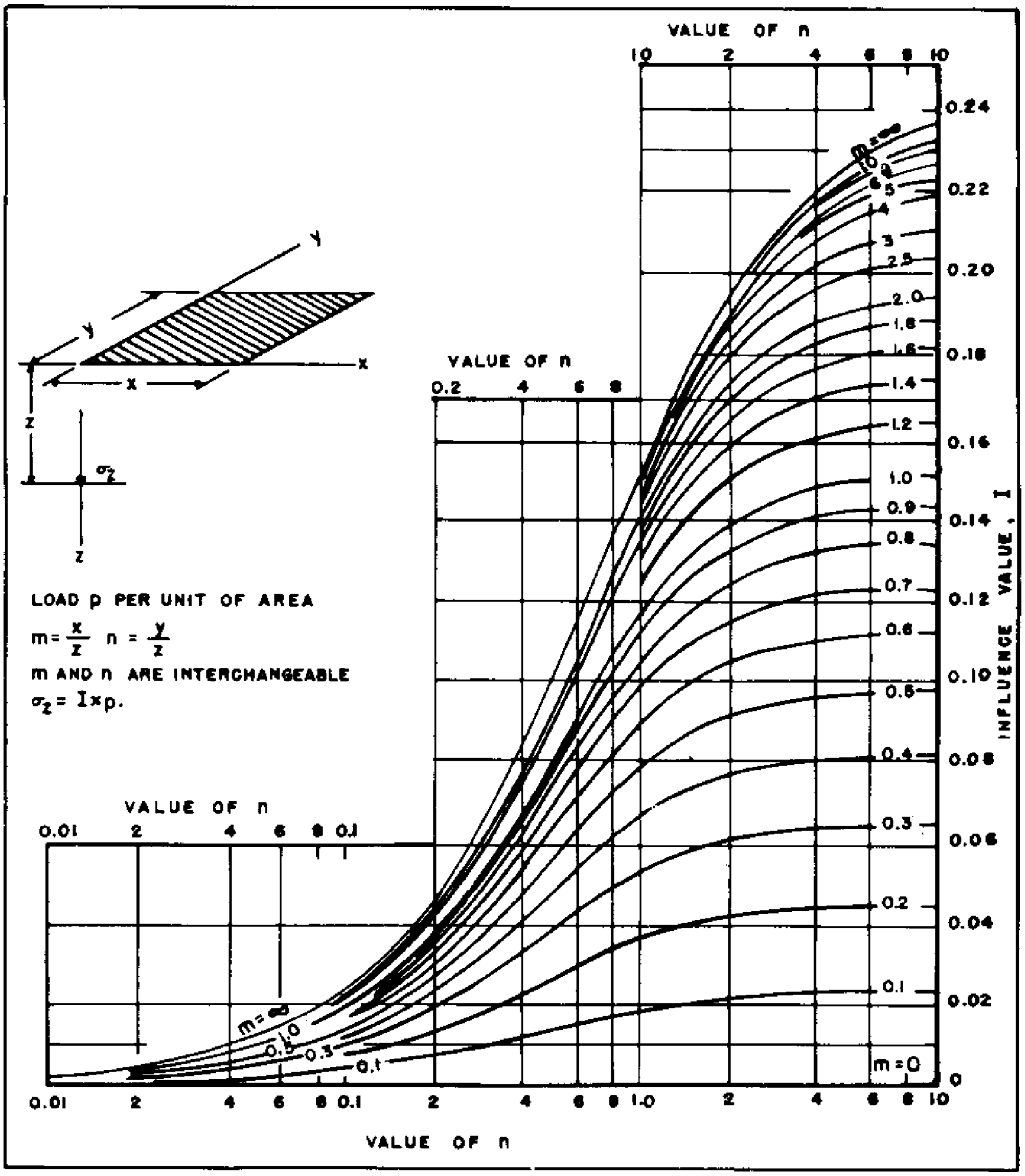


FIGURE 12
Influence Value for Vertical Stress Beneath a Corner of a
Uniformly Loaded Rectangular Area (Westergaard Case)

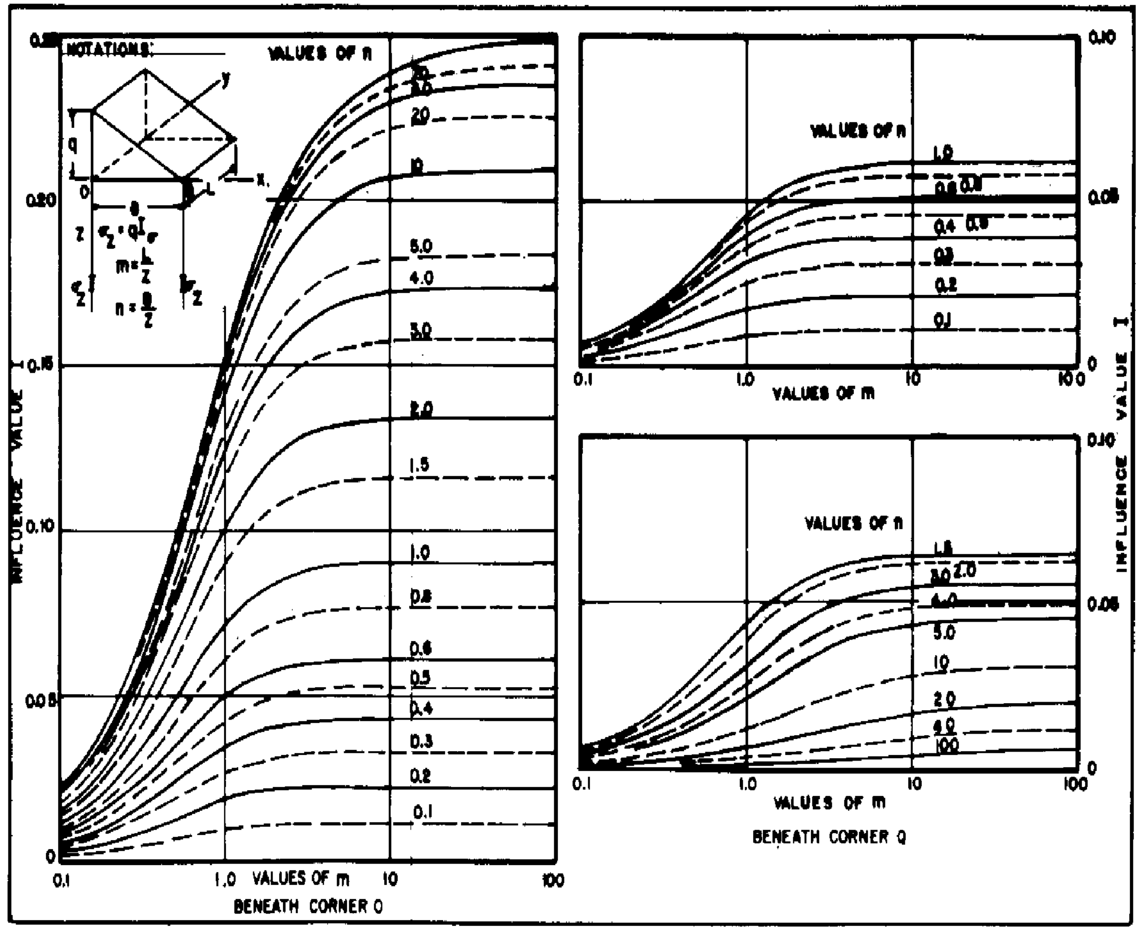


FIGURE 13
Influence Value for Vertical Stress Beneath Triangular Load
(Westergaard Case)

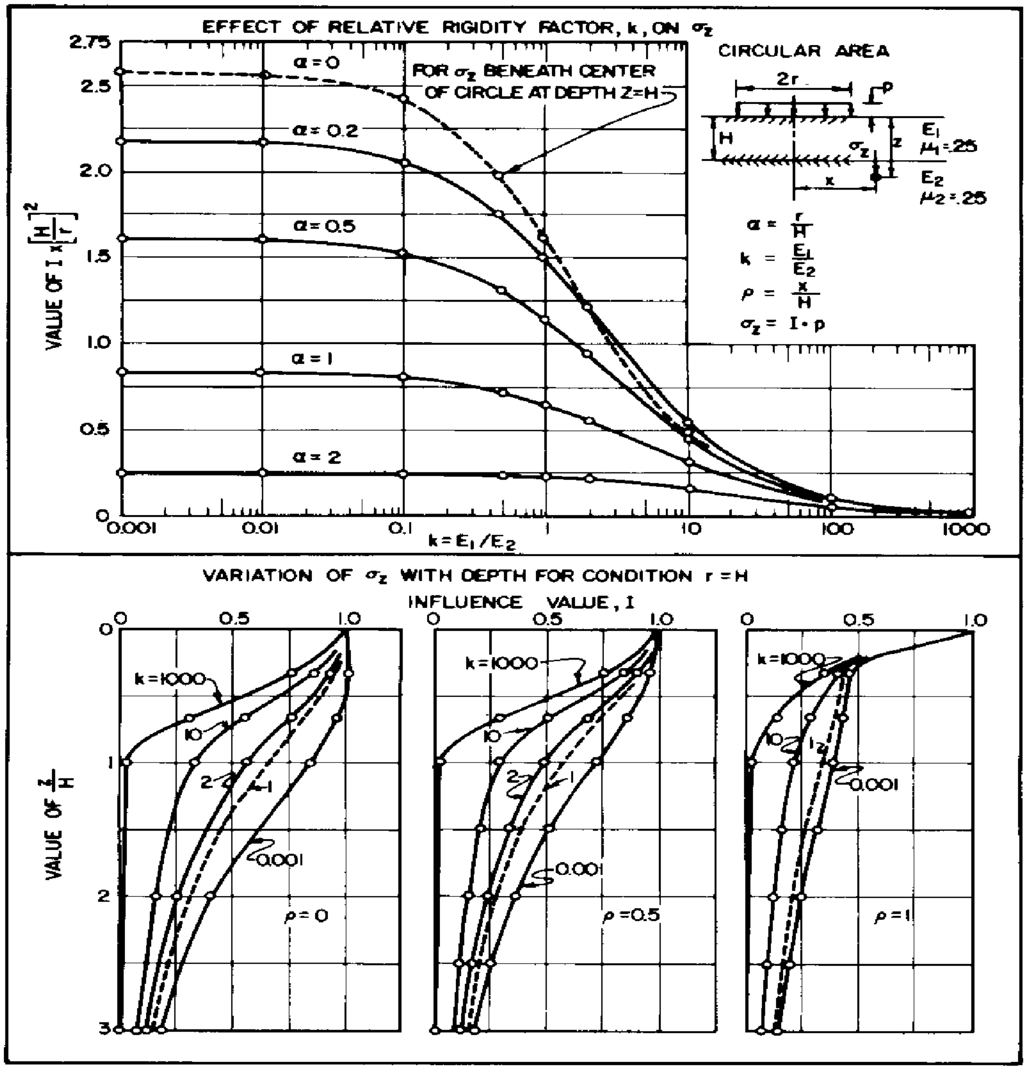
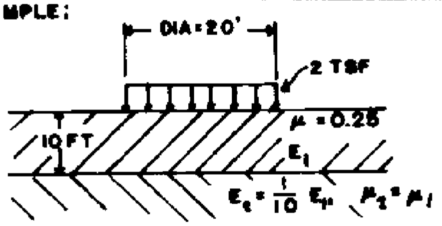


FIGURE 14
Influence Values for Vertical Stresses Beneath Uniformly Loaded Circular Area (Two-Layer Foundation)

EXAMPLE:



1. DETERMINE PROFILE OF STRESS INCREASE DUE TO APPLIED LOAD BELOW THE EDGES.

$a = \frac{10}{10} = 1$, $k = 10$

$\rho = \frac{10}{10} = 1$ — USE RIGHT HAND GRAPH OF LOWER PANEL OF FIGURE 14

DEPTH FT.	$\frac{Z}{H}$	I	$\sigma_z = I \cdot p$ TSF
5	0.5	0.35	0.70
10	1.0	0.21	0.42
15	1.5	0.15	0.30
20	2.0	0.12	0.24
25	2.5	0.10	0.20
30	3.0	0.07	0.14

FIGURE 15
Stress Profile in a Two-Layer Soil Mass

(1) Rigid Surface Layer Over Weaker Underlying Layer. If the surface layer is the more rigid, it acts as a distributing mat and the vertical stresses in the underlying soil layer are less than Boussinesq values.

(2) Weaker Surface Layer Over Stronger Underlying Layers. If the surface layer is less rigid than the underlying layer, then vertical stresses in both layers exceed the Boussinesq values. For influence diagrams for vertical stresses beneath rectangular loaded areas, see Reference 8, Stress and Displacement Characteristics of a Two-Layer Rigid Base Soil System: Influence Diagrams and Practical Applications, by Burmister. Use these influence diagrams to determine vertical stress distribution for settlement analysis involving a soft surface layer underlain by stiff material.

(3) Multi-Layer (Three or More) Systems. See Reference 6 for a discussion of the use of various approximate solutions for multi-layer systems.

c. Critical Depth. If there is no distinct change in the character of subsurface strata within the critical depth, elastic solutions for layered foundations need not be considered. Critical depth is the depth below the foundation within which soil compression contributes significantly to surface settlements. For fine-grained compressible soils, the critical depth extends to that point where applied stress decreases to 10 percent of effective overburden pressure. In coarse-grained material critical depth extends to that point where applied stress decreases to 20 percent of effective overburden pressure.

3. RIGID LOADED AREA. A rigid foundation must settle uniformly. When such a foundation rests on a perfectly elastic material, in order for it to deform uniformly the load must shift from the center to the edges, thus resulting in a pressure distribution which increases toward the edges (see Figure 16). This is the case for clays. In the case of sands, the soil near the edges yields because of the lack of confinement, thus causing the load to shift toward the center.

4. STRESSES INDUCED BY PILE LOADS. Estimates of the vertical stresses induced in a soil mass by an axially loaded pile are given in Figure 17 (Reference 9, Influence Scale and Influence Chart for the Computation of Stresses Due, Respectively, to Surface Point Load and Pile Load, by Grillo) for both friction and end-bearing piles. (See DM-7.2, Chapter 5 for further guidance on pile foundations.)

Section 4. SHALLOW PIPES AND CONDUITS

1. GENERAL. Pressures acting on shallow buried pipe and conduits are influenced by the relative rigidity of the pipe and surrounding soil, depth of cover, type of loading, span (maximum width) of structure, method of construction, and shape of pipe. This section describes simple procedures for determining pressures acting on a conduit in compressible soil for use in conduit design. For detailed analysis and design procedures for conduits in backfilled trenches and beneath embankments, consult one of the following:

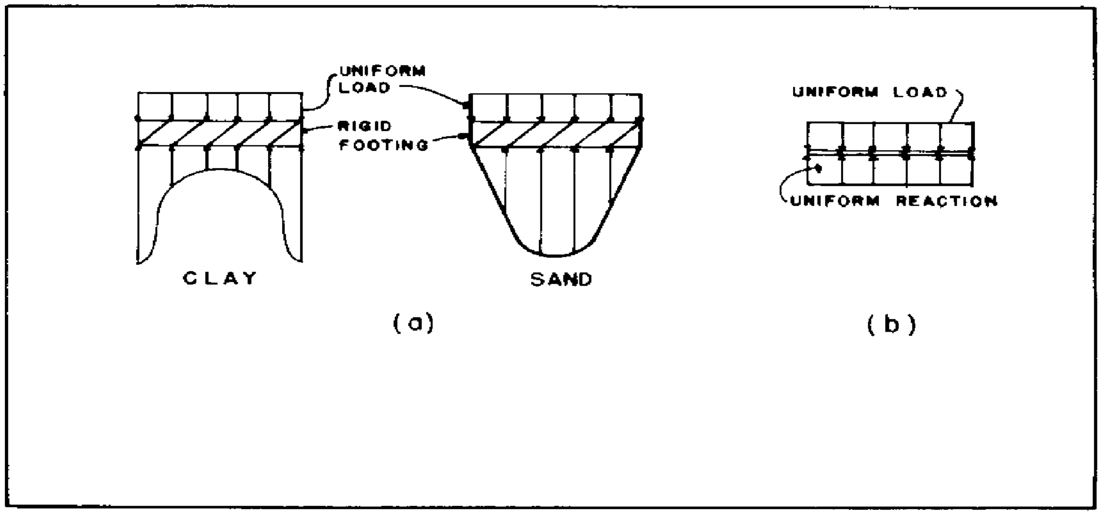


FIGURE 16
Contact Pressure Under (a) Rigid Footings
(b) Flexible Foundation on an Elastic Half Space

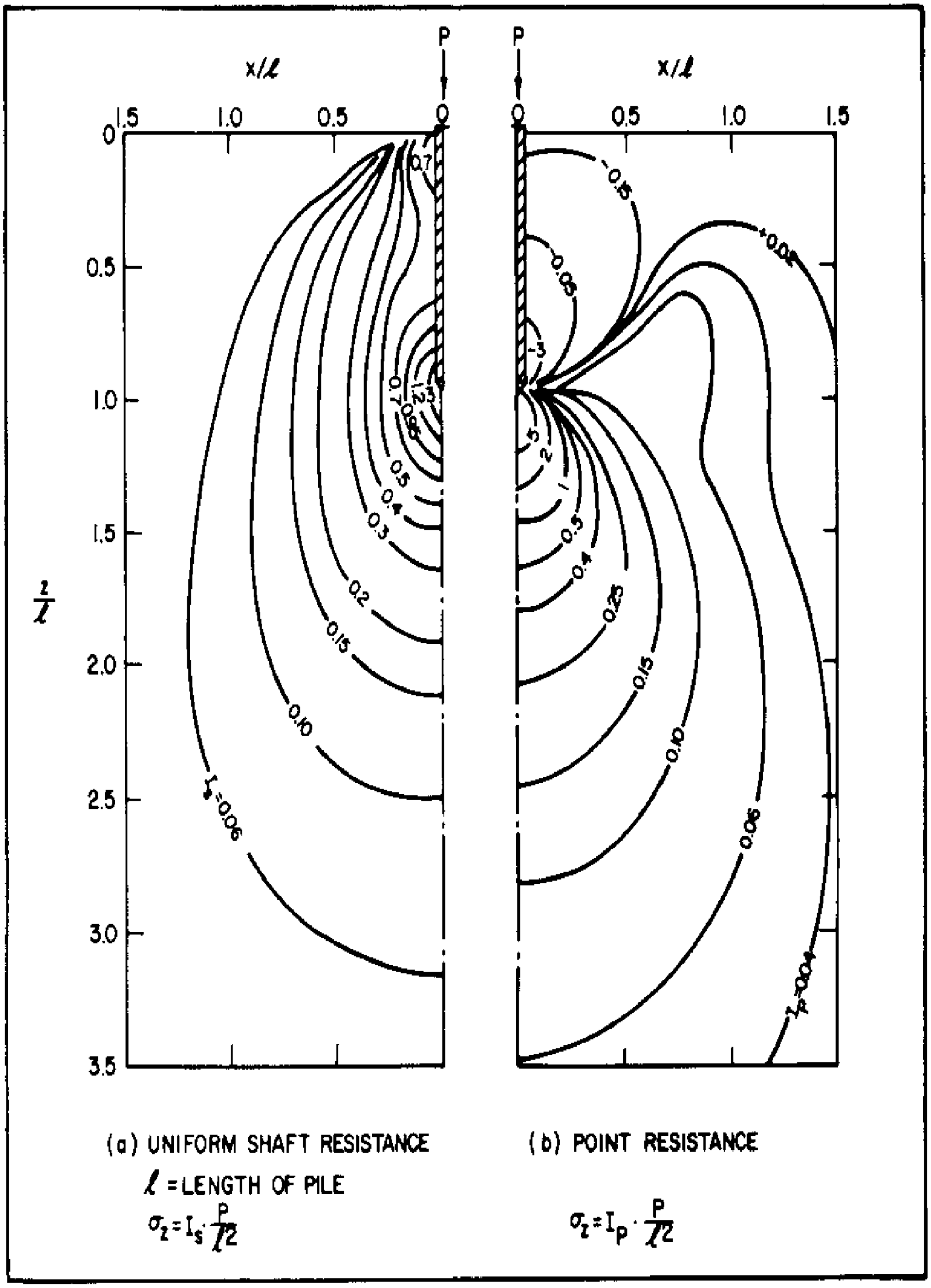


FIGURE 17

Influence Values for Vertical Stresses Around a Pile in an Elastic Solid

Reference 10, Buried Structures, by Watkins; Reference 11, Design and Construction of Sanitary and Storm Sewers, by the American Society of Civil Engineers; Reference 12, Handbook of Drainage and Construction Products, by Armco Drainage and Metal Products, Inc.; Reference 13, Engineering Handbook, Structural Design, by the U.S. Department of Agriculture, Soil Conservation Service; Reference 14, Concrete Pipe Design Manual, by American Concrete Pipe Association; or Reference 15, CANDE User Manual, by Katona and Smith.

2. RIGID PIPE. Pipes made from precast or cast-in-place concrete, or cast iron are considered rigid pipes.

a. Vertical Loads.

(1) Dead Load. Vertical soil pressure estimates for dead loads are obtained as follows:

EQUATION: W = C+w, .[Upsilon]-B.2- (4-1)

where W = total dead load on the conduit per unit length of conduit

C+w, = correction coefficient; function of trench depth to width ratio, angle of trench side slopes, friction angle of backfill and trench sides, bedding conditions

B = width of trench at level of top of pipe, or pipe outside diameter if buried under an embankment

[Upsilon] = unit weight of backfill

Dead load pressure, P+DL, = W) B

(a) Embankment Fill. Use Figure 18a (Reference 16, Underground Conduits - An Appraisal of Modern Research, by Spangler) to determine embankment dead load. For soils of unit weight other than 100 pcf, adjust proportionately; e.g., for [Upsilon] = 120 pcf, multiply chart by 1.20.

(b) Trench Backfill. Use Figure 18b (Reference 10) to determine values of C+w,.

(c) Jacked or Driven Into Place. Use Figure 18c (Reference 17, Soft Ground Tunneling, by Commercial Shearing, Inc.) for C+w,. This diagram may also be used for jacked tunnels.

(2) Live Load. Vertical pressure due to surface load, P+LL,, is calculated by Boussinesq equation (see Figure 2). Impact factor is included in the live load if it consists of traffic load. For example, an H-20 truck loading consists of two 16,000 lb. loads applied to two 10- by 20-inch areas. One of these loads is placed over the point in question, the other is 6 feet away. The vertical stresses produced by this loading including the effect of impact are shown in Figure 19 for various heights of cover.

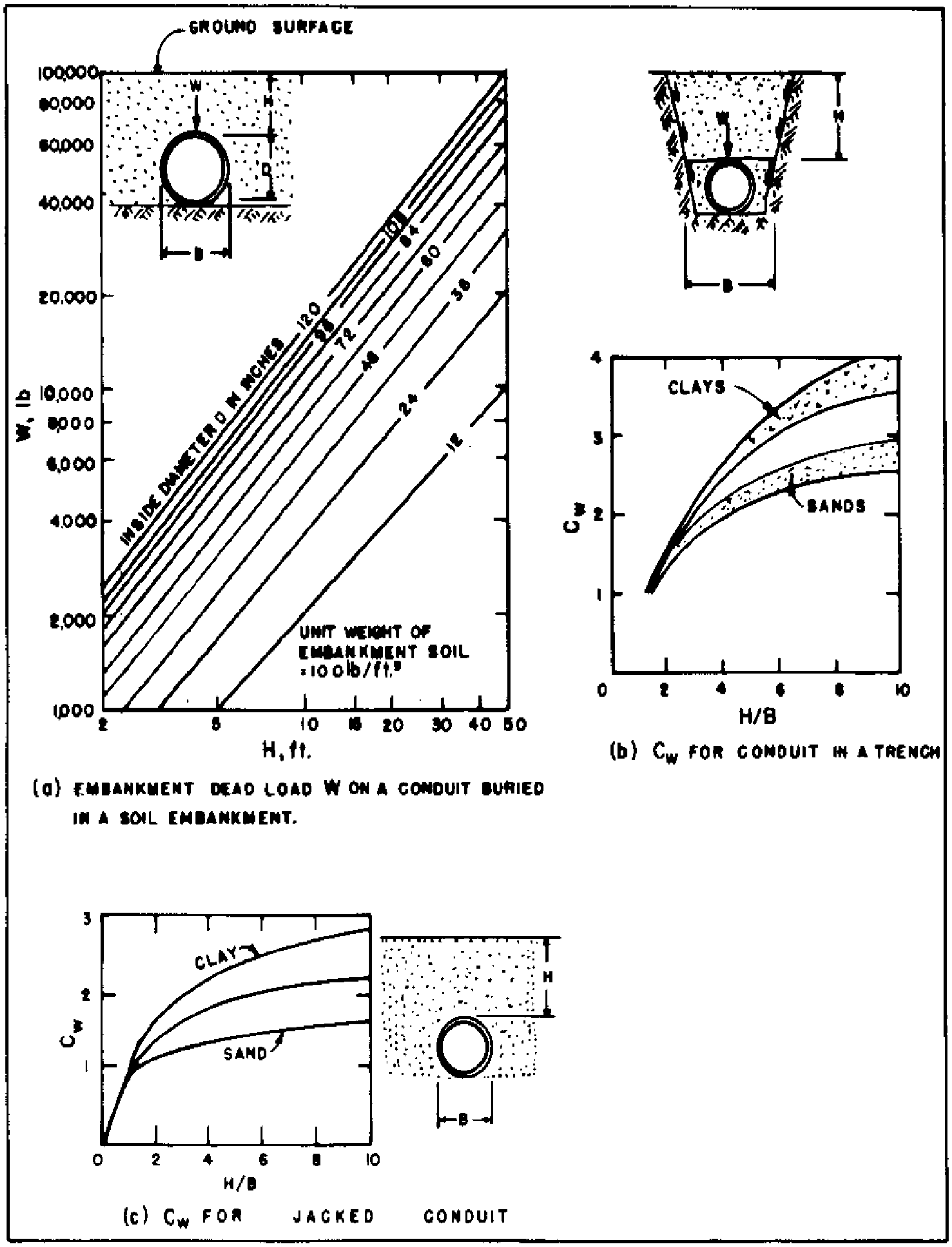
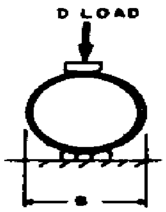


FIGURE 18
Backfill Coefficients, Embankment Loads, and Load Factors
for Rigid Conduits

(d) THREE EDGE BEARING METHOD.



(e) LOAD FACTORS L_f FOR RIGID PIPES BASED ON SPECIFIED CLASSES OF BEDDING.

CLASS A-CONCRETE CRADLE; B-COMPACTED GRANULAR MATERIAL; C-COMPACTED GRANULAR MATERIAL OR DENSELY COMPACTED BACKFILL; D-FLAT SUBGRADE.

	CLASS-A	CLASS-B	CLASS-C	CLASS-D
TRENCH[a]	4.8	1.9	1.5	1.1
"	3.4			
"	2.8			

[a] 4.8 FOR 1.0% REINFORCING STEEL; 3.4 FOR 0.4% REINFORCING STEEL; 2.8 FOR PLAIN.

FIGURE 18 (continued)
Backfill Coefficients, Embankment Loads, and Load Factors for Rigid Conduits

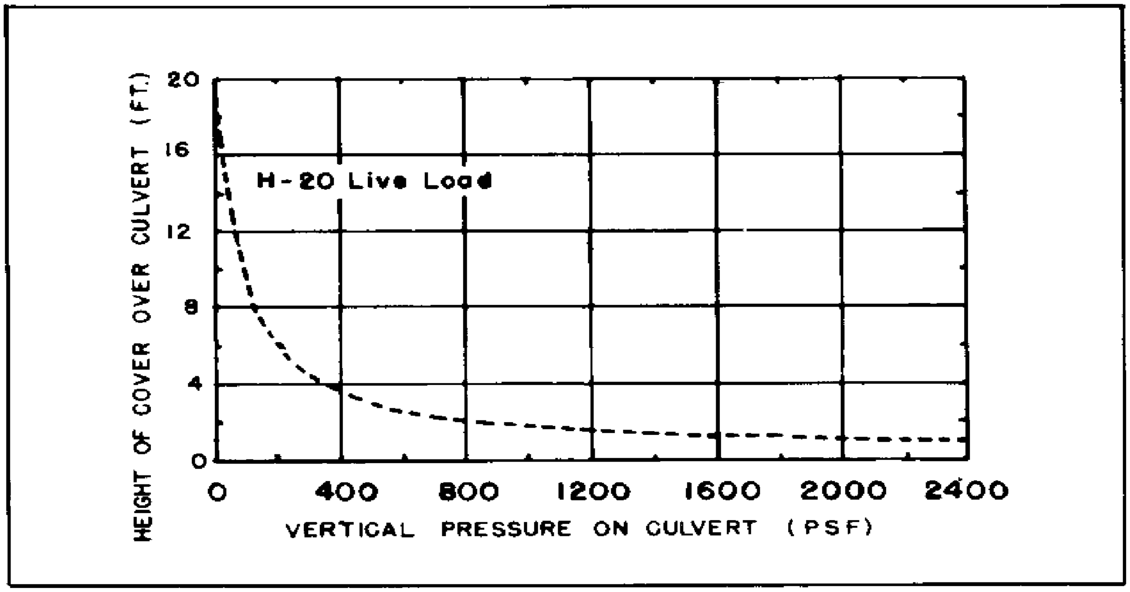


FIGURE 19
Vertical Pressure on Culvert Versus Height of Cover

b. Design of Rigid Conduit. To design a rigid conduit, the computed loads (dead and live) are modified to account for bedding conditions and to relate maximum allowable load to the three-edge bearing test load D. (See Figure 18d.) See ASTM C76, Reinforced Concrete Culvert, Storm Drain, and Sewer Pipe, for test standards for D load.

Bedding conditions for pipes in trenches may be accounted for by use of a load factor, L_f. Determine L_f from Figure 18e (Reference 14). Determine D from the following equation:

EQUATION: $$D_{0.01} = (P_{DL} + P_{LL}) \frac{N}{L_f} \qquad (4\text{-}2)$$

where $D_{0.01}$ = Allowable load in lb/ft of length of conduit per foot of inside diameter for a crack width of 0.01"

L_f = load factor

N = safety factor (usually 1.25)

With the specified D load, the supplier is able to provide adequate pipe.

The soil pressure against the sides of a pipe in an embankment significantly influence the resistance of the pipe to vertical load. The load factor for such cases considers not only pipe bedding, but also pipe shape, lateral earth pressure, and the ratio of total lateral pressure to total vertical pressure. For further guidance see Reference 11.

3. FLEXIBLE STEEL PIPE. Corrugated or thin wall smooth steel pipes are sufficiently flexible to develop horizontal restraining pressures approximately equal to vertical pressures if backfill is well compacted. Vertical exterior pressure acting at the top of the pipe may range from pressures exceeding overburden pressure in highly compressible soils to much less than the overburden pressure in granular soils because of the effect of "arching", in which a portion of the overburden pressure is supported by the surrounding soil.

a. Vertical Loads.

(1) Dead Load. For flexible pipe, the dead load pressure is simply the height of the column of soil above the conduit times the unit weight of the backfill, as follows:.

EQUATION: $$P_{DL} = \Upsilon \times H \qquad (4\text{-}3)$$

(2) Live Load. Computed by Boussinesq equations for rigid pipes.

(3) Pressure Transfer Coefficient. The dead load and live load pressures are modified by pressure transfer coefficient, C_p, to yield apparent pressure, P, to be used in design.

EQUATION: $$P = C_p(P_{LL} + P_{DL}) \qquad (4\text{-}4)$$

See Figure 20 (Reference 18, Response of Corrugated Steel Pipe to External Soil Pressures, by Watkins and Moser) for the values of C_p.

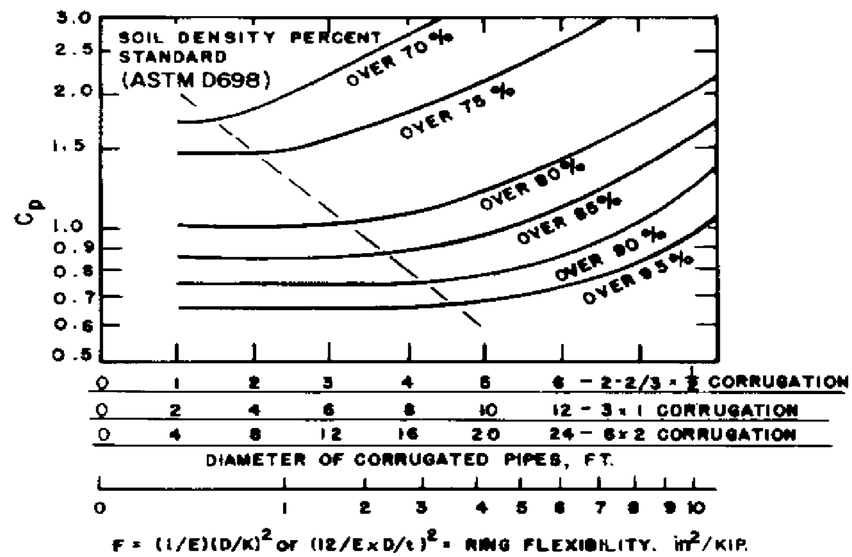


FIGURE 20
Pressure Transfer Coefficients for Corrugated Flexible Conduits as a Function of Standard Soil Density and Ring Flexibility or Diameter and Corrugation Depth

b. Initial Designs. Use the following design procedures:

(1) Determine apparent ring compression stress of the pipe:

EQUATION: Apparent ring comp. stress $= \frac{PD}{2A}$ (4-5)

where P = apparent vertical soil pressure on top of conduit, as determined from Equation (4-4)

D = outside diameter of conduit

A = cross-sectional area of the wall per unit length of conduit

(2) Equate apparent ring compression stress to allowable ring compression strength to determine required cross-sectional wall area, A, per unit length of pipe:

EQUATION: Allowable ring comp. strength $= \frac{S_y}{F_S}$ (4-6)

EQUATION: $A = \frac{PDS_y}{2F_S}$ (4-7)

where S_y = yield point strength of the steel (typically 33 to 45 ksi)

F_S = safety factor (usually 1.5 to 2)

(3) Select appropriate pipe size to provide the minimum cross-sectional wall area A as determined above.

(4) Check ring deflection so that it does not exceed 5% of the nominal diameter of the pipe. Ring deflection Y, is governed by the total soil pressure $P_v = P_{DL} + P_{LL}$, diameter D, moment of inertia I, modulus of elasticity of conduit E, and soil modulus E'. Generally, ring deflection does not govern the design. See Figure 21 (Reference 10) for an example.

(5) The Handling Factor is the maximum flexibility beyond which ring is easily damaged. Pipe design must consider limiting the Handling Factor to such typical values as D^2/EI = 0.0433 in/lb for 2-2/3 x 1/2 corrugation and 0.0200 in/lb for 6 x 2 corrugation.

c. Soil Placement. Great care must be exercised in soil placement. Ring deflection and external soil pressures are sensitive to soil placement. If a loose soil blanket is placed around the ring and the soil is carefully compacted away from it, soil pressure is reduced considerably.

d. Design of Flexible Steel Pipe. For analysis and design procedures for large size flexible pipe of non-circular cross section, see Reference 12.

4. CONDUITS BENEATH EMBANKMENTS OF FINITE WIDTH. Design of culverts and conduits beneath narrow-crested embankments must consider the effect of the embankment base spread and settlement on the pipe.

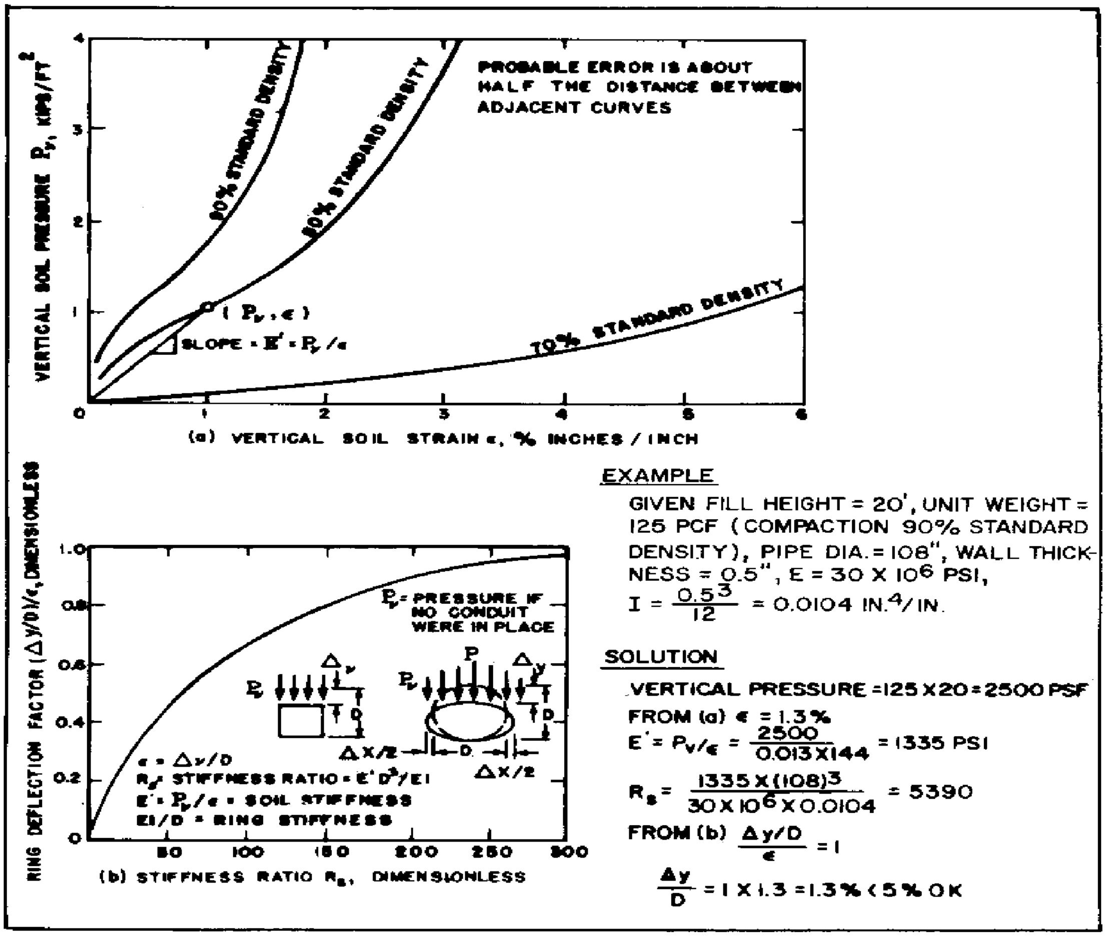


FIGURE 21
Example of Ring Deflection

a. Longitudinal Extension. The maximum horizontal strain of a conduit beneath an embankment or earth dam occurs under the center of the fill. Maximum strain depends on the ratios b/h, b/d, and the average vertical strain in the foundation beneath center of the fill. (See Figure 22 for the definitions and the relationship between vertical strain and horizontal strain.)

b. Joint Rotation. Besides the horizontal extension of the conduit, additional joint opening may occur at the bottom of the pipe because of settlement under the embankment load. For concrete pipe in sections about 12 feet long, compute additional joint opening due to settlement by Equation (4-8).

EQUATION:
$$\text{Opening} = \frac{\delta c r}{b} \qquad (4\text{-}8)$$

where δ = settlement of base of pipe at embankment centerline (in)

b = embankment base width (in)

c = constant, varying from 5 for uniform foundation conditions to 7 for variable foundation conditions

r = pipe radius (in)

c. Pipe Selection. Compute total settlement below embankment by methods in Chapter 5. From this value, compute maximum joint opening at pipe mid-height as above. Add to this opening the spread at the top or bottom of the pipe from joint rotation computed from Equation (4-8).

Specify a pipe joint that will accommodate this movement and remain watertight. If the joint opening exceeds a safe value for precast concrete pipe, consider cast-in-place conduit in long sections with watertight expansion joints. Corrugated metal pipe is generally able to lengthen without rupture, but it may not be sufficiently corrosion resistant for water retention structures.

5. LONG SPAN METAL CULVERTS. The above methods are not applicable to very large, flexible metal culverts, i.e., widths in the range of 25 to 45 feet. For analysis and design procedures for these see Reference 19, Behavior and Design of Long Span Metal Culverts, by Duncan.

Section 5. DEEP UNDERGROUND OPENINGS

1. GENERAL FACTORS. Pressures acting on underground openings after their completion depend on the character of the surrounding materials, inward movement permitted during construction, and restraint provided by the tunnel lining.

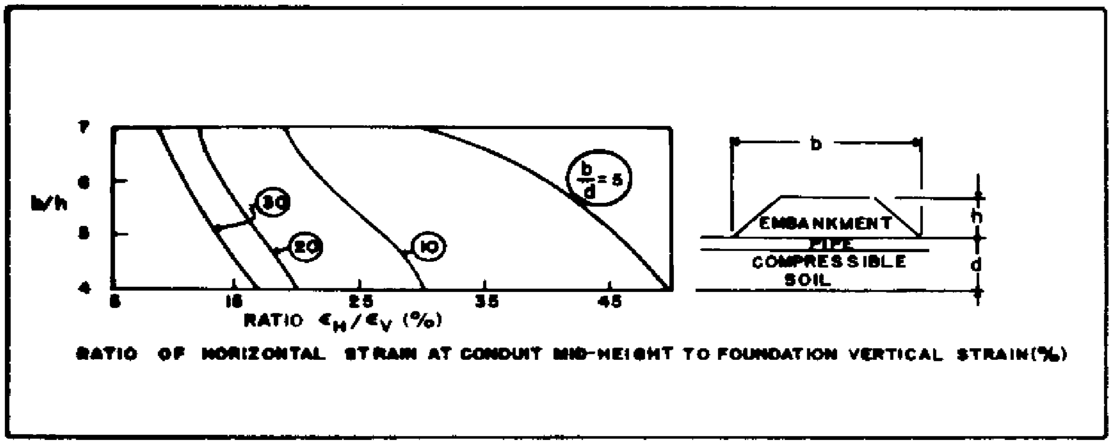


FIGURE 22
Conduits Beneath Embankments of Finite Width

2. OPENINGS IN ROCK. Stress analysis differs for two rock groups: sound, nonswelling rock that can sustain considerable tensile stresses, and fractured blocky, seamy, squeezing, or swelling rock. For detailed explanations of these rock groups, see Chapter 1.

a. Sound Rock. Determine stresses surrounding tunnels or openings in intact, isotropic rock, such as crystalline igneous types, or homogeneous sandstone and limestone, by elastic analyses. Use the methods of Reference 20, Design of Underground Openings in Competent Rock, by Obert, et al.

For these materials, stresses in rock surrounding spheroidal cavities are lower than those for tunnels with the same cross section. Use elastic analyses to determine the best arrangement of openings and pillars, providing supports as required at locations of stress concentrations. For initial estimates of roof pressure, Table 1 (Reference 21, Rock Tunneling with Steel Supports, by Proctor and White) may be used.

b. Broken and Fractured Rock. Pressure on tunnels in chemically or mechanically altered rock must be analyzed by approximate rules based on experience. For details, see Reference 21.

c. Squeezing and Swelling Rocks. Squeezing rocks contain a considerable amount of clay. The clay fraction may be from non-swelling kaolinite group or from highly swelling montmorillonite group. These rocks are preloaded clays and the squeezing is due to swelling. The squeeze is intimately related to an increase in water content and a decrease in shear strength.

3. LOADS ON UNDERGROUND OPENINGS IN ROCK.

a. Vertical Rock Load. Table 1 gives the height of rock above the tunnel roof which must be supported by roof lining.

b. Horizontal Pressures. Determine the horizontal pressure P+a, on tunnel sides by applying the surcharge of this vertical rock load to an active failure wedge (see diagram in Table 1). Assume values of rock shear strength (see Chapter 3 for a range of values) on the active wedge failure plane, which allow for the fractured or broken character of the rock. Evaluate the possibility of movement of an active failure plane that coincides with weak strata or bedding intersecting the tunnel wall at an angle.

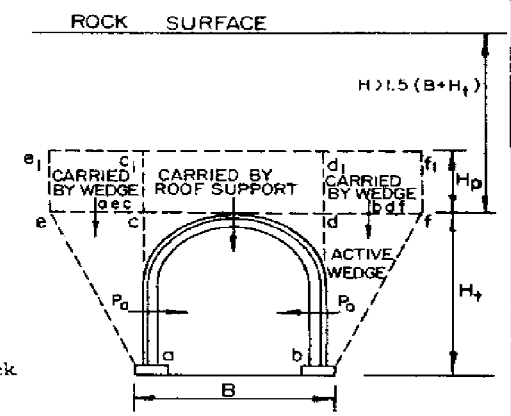
c. Support Pressures as Determined From Rock Quality. As an alternate method of analysis, use empirical correlations in Reference 22, Engineering Classification of Rock Masses for Tunnel Support, by Barton, et al., to determine required support pressures as a function of rock mass quality "Q". The analysis incorporates rock quality designation (RQD) and various joint properties of the surrounding material, and is applicable for sound or fractured rock. Results may be used directly for evaluating type of roof or wall support required.

TABLE 1
Overburden Rock Load Carried by Roof Support

Rock Conditions	Rock Load H_p in Feet	Remarks
1. Hard and intact	Zero	Sometimes spalling or popping occurs.
2. Hard stratified or schistose	0 to 0.5 B	Light pressures.
3. Massive, moderately jointed	0 to 0.25 B	Load may change erratically from point to point.
4. Moderately blocky and seamy	0.25 B to 0.35 $(B+H_t)$	No side pressure.
5. Very blocky and seamy	0.35 to 1.10 $(B+H_t)$	Little or no side pressure.
6. Completely crushed but chemically intact	1.10 $(B+H_t)$	Considerable side pressure. Softening effect of seepage towards bottom of tunnel.
7. Squeezing rock, moderate depth	(1.10 to 2.10) $(B+H_t)$	Heavy side pressure.
8. Squeezing rock, great depth	(2.10 to 4.50) $(B+H_t)$	
9. Swelling rock	Up to 250 ft. irrespective of value of $(B+H_t)$	Very heavy pressures.

Notes:

1. Above values apply to tunnels at depth greater than 1.5 $(B+H_t)$.
2. The roof of the tunnel is assumed to be located below the water table. If it is located permanently above the water table, the values given for rock conditions 4 to 6 can be reduced by fifty percent.
3. Some very dense clays which have not yet acquired properties of shale rock may behave as squeezing or swelling rock.
4. Where sandstone or limestone contain horizontal layers of immature shale, roof pressures will correspond to rock condition "very blocky and seamy."



4. OPENINGS IN SOFT GROUND.

a. Ground Behavior. The method of construction of tunnels depends upon the response of the ground during and after excavations. The stand up time depends upon the type of soil, the position of groundwater, and the size of opening. Depending upon the response during its movement period, the ground is classified as: (1) firm, (2) raveling, (3) running, (4) flowing, (5) squeezing or (6) swelling.

(1) In firm ground, no roof support is needed during excavation and there is no perceptible movement.

(2) In raveling ground, chunks or flakes of material begin to fall prior to installing the final ground supports. Stand up time decreases with increasing size of excavation. With rising groundwater, raveling ground may become running ground. Sand with clay binder is one example of this type of soil.

(3) In running ground, stand up time is zero. The roof support must be inserted prior to excavation. Removal of side supports results in inflow of material which comes to rest at its angle of repose. Dry cohesionless soils fall into this category.

(4) Flowing ground acts as a thick liquid and it invades the opening from all directions including the bottom. If support is not provided, flow continues until the tunnel is completely filled. Cohesionless soil below groundwater constitutes flowing ground.

(5) Squeezing ground advances gradually into the opening without any signs of rupture. For slow advancing soil, stand up time is adequate, yet the loss of ground results in settlement of the ground surface. Soft clay is a typical example of squeezing ground.

(6) Swelling ground advances into the opening and is caused by an increase in volume due to stress release and/or moisture increase. Pressures on support members may increase substantially even after the movement is restrained.

b. Loss of Ground. As the underground excavation is made, the surrounding ground starts to move toward the opening. Displacements result from stress release, soil coming into the tunnel from raveling, runs, flows, etc. The resulting loss of ground causes settlement of the ground surface. The loss of ground associated with stress reduction can be predicted reasonably well, but the ground loss due to raveling, flows, runs, etc. requires a detailed knowledge of the subsurface conditions to avoid unacceptable amounts of settlement. For acceptable levels of ground loss in various types of soils see Reference 23, Earth Tunneling with Steel Supports, by Proctor and White.

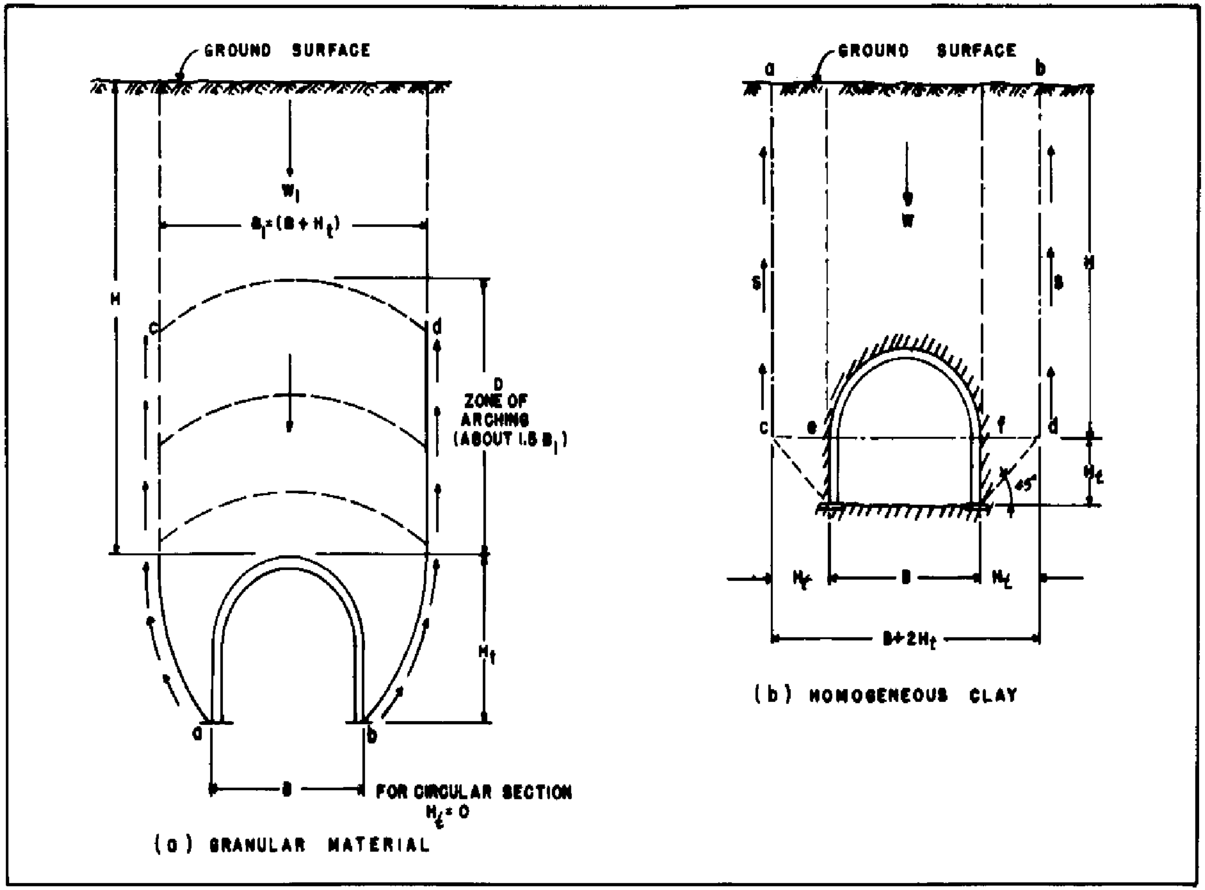


FIGURE 23
Load Action on Underground Openings in Earth

c. <u>Loads</u>. The support pressures in the underground openings are governed by the unit weight of the soil, groundwater table, soil properties, deformations during excavation, interaction between soil and the supports, shape of the opening, and the length of time that has elapsed since the installation of lining. Other factors such as the presence of another opening adjacent to it, excavation of a large deep basement near an existing opening, load from neighboring structures, and change in groundwater conditions, will also affect the design pressures on the tunnel supports. A schematic representation of the load action on underground openings is shown in Figure 23 (Reference 23).

Estimate of load for temporary supports in earth tunnels may be obtained from Table 2 (Reference 23). For further guidance see Reference 23 and Reference 24, <u>Tunneling in Soft Ground, Geotechnical Considerations</u>, by Peck.

5. PRESSURE ON VERTICAL SHAFTS.

a. <u>Shaft in Sand</u>. In the excavation of a vertical cylindrical shaft granular soils, pressures surrounding the shaft approach active values. If outward directed forces from a buried silo move the silo walls into the surrounding soil, pressures approach passive values as an upper limit.

(1) Pressure Coefficients. See Figure 24 for active and passive pressure coefficients for a cylindrical shaft of unlimited depth in granular soils.

(2) Modification of Active Pressures. For relatively shallow shafts (depth less than twice the diameter), rigid bracing at the top may prevent development of active conditions. In this case, horizontal pressures may be as large as at-rest pressures on a long wall with plane strain in the surrounding soil. (See DM-7.2, Chapter 3.)

(3) If groundwater is encountered, use submerged unit weight of sand and add hydrostatic pressure.

b. <u>Shaft in Clay</u>.

(1) Pressure on Walls of Shafts in Soft Clay. For a cylindrical shaft, no support is needed from the ground surface to a depth of $z+o, = \frac{2C}{[Upsilon]}$. To determine the approximate value of ultimate horizontal earth pressure on a shaft lining at any depth z, use

ph = [Upsilon] [multiplied by] z-c

where [Upsilon] = effective unit weight of clay

z = depth

c = cohesion

This pressure is likely to occur after several months.

TABLE 2
Loads For Temporary Supports in Earth Tunnels at Depths More Than 1.5 (B + H+t,)

Type of Ground	Ground Condition	Design Load[*] H+p,	Remarks
Running ground above water table	Loose	0.50 (B + H+t,)	
	Medium	0.04 (B + H+t,)	
	Dense	0.30 (B + H+t,)	
Running ground in compressed-air tunnel		Disregard air pressure; H+p, equal to that for running ground, above water table with equal density.	
Flowing ground in free-air tunnel		H or 2 (B + H+t,) whichever is smaller	
Raveling ground	Above water table	+ , *T-t * *))) * H+p, (running) * T * . -	
	Below water table (free air)	+ , *T-t * *))) * H+p, (running) * T * . -	
	Below water table (compressed air)	+ , *T-t * P+c, *))) * 2H+p, -))))))) * T * [Upsilon] . -	
Squeezing ground	Homogeneous	P+c, Hq+u, H -))))))) -))))))))))))))))) [Upsilon] 2[Upsilon](B +2H+t,)	After complete blowout, P+c, = 0
	Soft roof, stiff sides	P+c, Hq+u, H -))))))) -)))))))))))))))))) [Upsilon] 2[Upsilon] B	
	Stiff roof, soft sides	P+c, Hq+u, H -))))))) -))))))))))))))))) [Upsilon] 2[Upsilon](B +6H+t,)	

TABLE 2 (continued)
Loads For Temporary Supports in Earth Tunnels at Depths More Than 1.5 (B + H+t,)

Type of Ground	Ground Condition	Design Load[*] H+p,	Remarks
Swelling ground	Intact	Very small	Permanent roof support should be completed within a few days after mining
	Fissured, above water table	H+p, equal to that for raveling ground with same stand up time H	
	Fissured, below water table, free-air tunnel		

p+c, = air pressure in pounds per square foot

q+u, = unconfined compressive strength of ground above roof in pounds per square foot

[Upsilon] = unit weight of soil in pounds per cubic foot

t = stand up time, minutes

T = elapsed time between excavating and completion of permanent structure, minutes

H = vertical distance between ground surface and tunnel roof in feet

H+p, = design load in feet of earth, see Table 1

H+t, = height of tunnel, see Table 1

B = width of tunnel, see Table 1

[*] For circular tunnels, H+t, = 0, B = Diameter

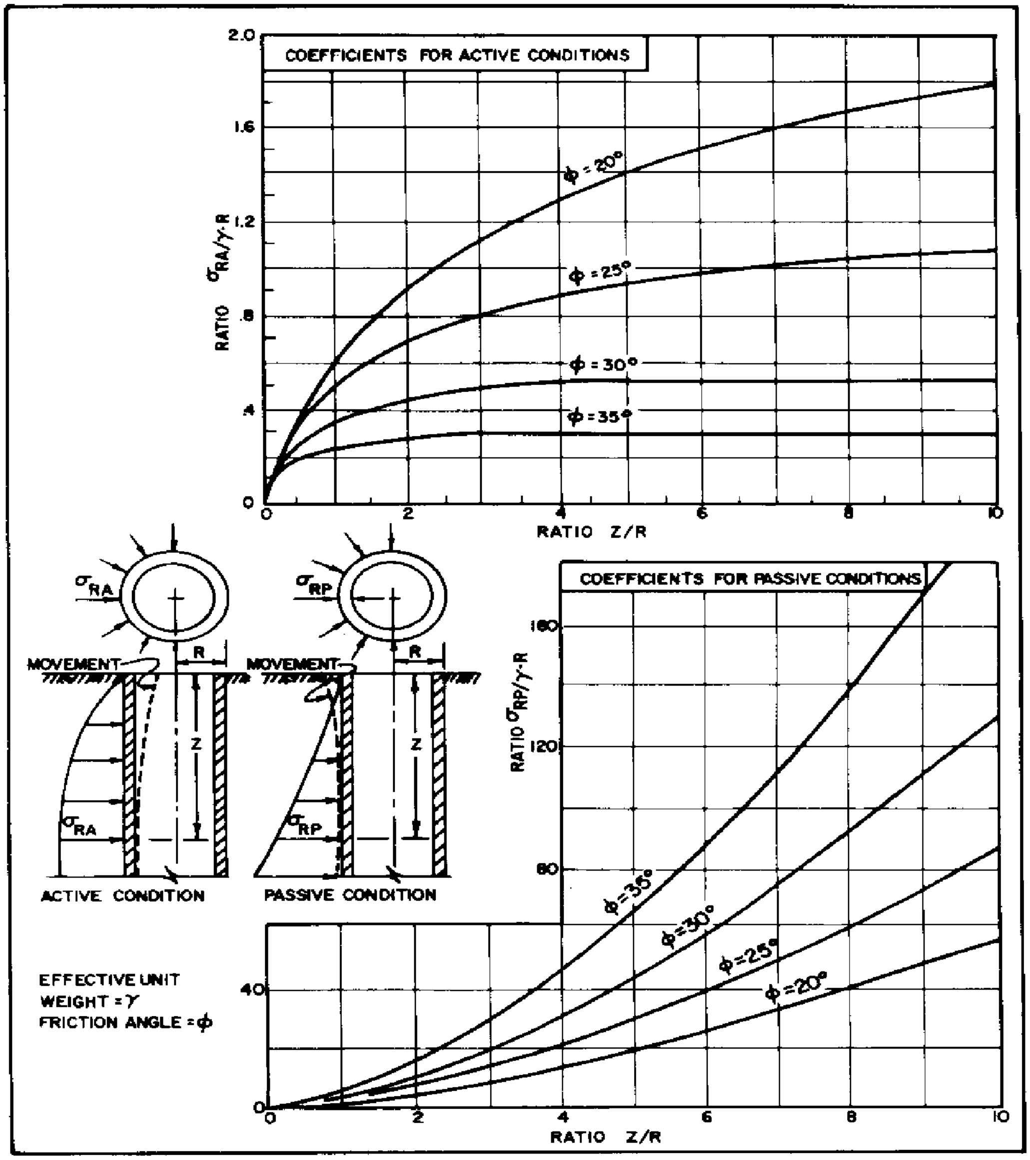


FIGURE 24
Coefficients for Active or Passive Pressures on Underground
Cylindrical Shafts or Silos

(2) Pressure on Walls of Shafts in Stiff Clay. On shafts located in stiff, intact, or fissured swelling clays, initially the pressure on the shaft lining is very small. Over a period of time, the pressure may increase to several times the overburden pressure (i.e., ultimately to the swelling pressure if shaft lining is sufficiently rigid). Local experience in that soil or field measurements can provide useful information. For further details of pressures on shafts, see Reference 23.

Section 6. NUMERICAL STRESS ANALYSIS

Stress analysis using numerical methods and computers are available for many simple as well as more complex loading conditions. See DM-7.3, Chapter 3 on available computer programs.

REFERENCES

1. Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, Stresses and Deflections in Foundations and Pavements, Fall, 1965.

2. Foster, C.R., and Ahlvin, P.G., Stresses and Deflections Induced by Uniform Circular Load, Highway Research Board Proceedings, Highway Research Board, Washington, DC, 1954.

3. Osterberg, J.O., Influence Values for Vertical Stresses in a Semi-Infinite Mass Due to an Embankment Loading, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

4. Jimenez Salas, J.A., Soil Pressure Computations: A Modification of Newmark's Method, Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, 1948.

5. Poulos, H.G. and Davis, E.H., Elastic Solutions for Soil and Rock Mechanics, John Wiley & Sons, Inc., New York, 1974.

6. Duncan, J.M., and Buchignani, A.L., An Engineering Manual for Settlement Studies, Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, June, 1976.

7. Mehta, M.R., and Veletsos, A.S., Stresses and Displacement in Layered Systems, Structural Research Series No. 178, University of Illinois, Urbana, IL.

8. Burmister, D.M., Stress and Displacement Characteristics of a Two-Layer Rigid Base Soil System: Influence Diagrams and Practical Applications, Proceedings, Highway Research Board, Washington, DC, 1956.

9. Grillo, O., Influence Scale and Influence Chart for the Computation of Stresses Due, Respectively, to Surface Point Load and Pile Load, Proceedings of Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam, Vol. 6, pp 70-73, 1948.

10. Watkins, R.K., Buried Structures, Foundation Engineering Handbook, Winterkorn H.F. and Fang, F.Y., ed., Chapter 23, Van Nostrand and Reinhold Co., New York, 1975.

11. American Society of Civil Engineers, Design and Construction of Sanitary and Storm Sewers, Manuals and Reports of Engineering Practice, No. 37, ASCE, 1976.

12. Armco Drainage and Metal Products, Inc., Handbook of Drainage and Construction Products.

13. U. S. Department of Agriculture, Soil Conservation Service, Engineering Handbook, Structural Design, Section 6.

14. American Concrete Pipe Association, Concrete Pipe Design Manual, Vienna, VA, 1980.

15. Katona, M.G. and Smith, J.M., CANDE User Manual, FHWA-RD-77-6, Federal Highway Administration, October, 1976.

16. Spangler, M.G, Underground Conduits - An Appraisal of Modern Research, Transactions, ASCE, pp 316-373, 1948.

17. Commercial Shearing, Inc., Soft Ground Tunneling (company brochure), Catalog T-1, Youngstown, OH, 1971.

18. Watkins, R.K. and Moser, A.P., Response of Corrugated Steel Pipe to External Soil Pressures, Highway Research Record Number 373, Highway Research Board, 1971.

19. Duncan, J.M., Behavior and Design of Long Span Metal Culverts, Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT3, 1979.

20. Obert, L., Duvall, W.I. and Merrill, R.H., Design of Underground Openings in Competent Rock, Bulletin, U.S. Bureau of Mines.

21. Proctor, R.V. and White, T.L., Rock Tunneling With Steel Supports, Commercial Shearing Inc., Youngstown, OH, 1968.

22. Barton, N., Lien, R. and Lunde, J., Engineering Classification of Rock Masses for Tunnel Support, Rock Mechanics, Volume 6, No. 4, Journal International Society of Rock Mechanics, 1974.

23. Proctor, R.V. and White, T.L., Earth Tunneling With Steel Supports, Commercial Shearing, Inc., Youngstown, OH, 1977.

24. Peck, R.B., Tunneling in Soft Ground, Geotechnical Considerations, Seminar on Underground Construction, Vail, CO, 1976.

25. Naval Facilities Engineering Command, Design Manuals (DM)

DM-5.03	Drainage Systems
DM-21 Series	Airfield Pavements

Copies of design manuals may be obtained,from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, Pennsylvania 19120.

CHAPTER 5. ANALYSIS OF SETTLEMENT AND VOLUME EXPANSION

Section 1. INTRODUCTION

1. SCOPE. This chapter concerns (a) immediate settlements, (b) long-term settlements, (c) rate of settlement, (d) criteria for tolerable settlement, (e) methods of reducing or accelerating settlements for saturated fine-grained soils and (f) methods for controlling and/or estimating heave in swelling soils. Procedures given are for fine-grained compressible soils as well as for coarse-grained soils.

Guidance in other special cases such as collapsing soil, sanitary land fill, etc., is provided in DM-7.3, Chapter 3. Monitoring of settlements is discussed in Chapter 2.

2. OCCURRENCE OF SETTLEMENTS. The settlement of saturated cohesive soil consists of the sum of three components; (1) immediate settlement occurring as the load is applied, (2) consolidation settlement occurring gradually as excess pore pressures generated by loads are dissipated, and (3) secondary compression essentially controlled by the composition and structure of the soil skeleton.

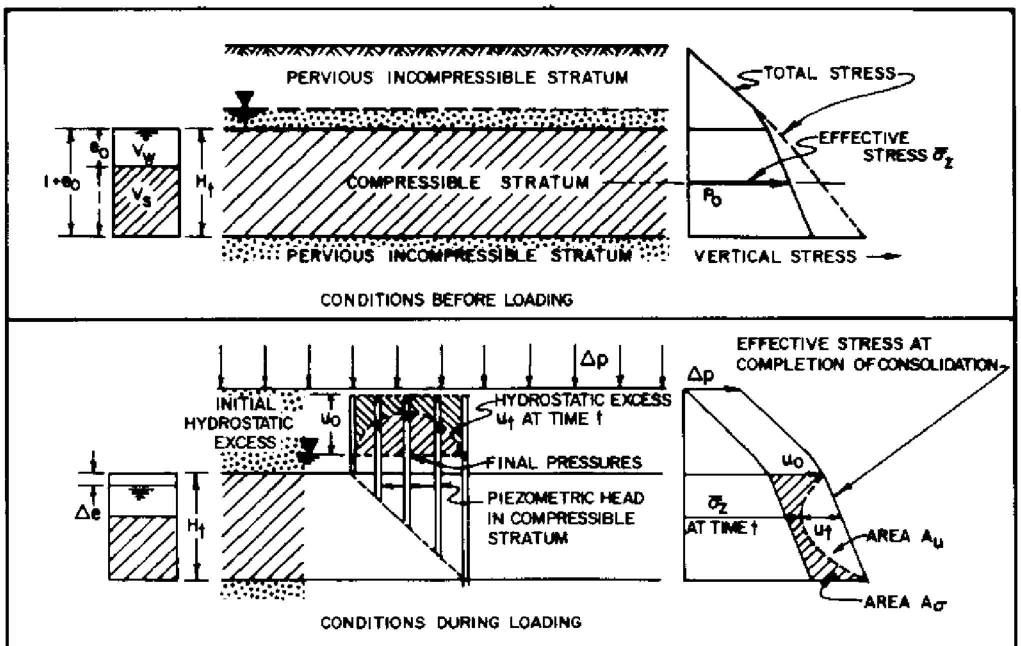
The settlement of coarse-grained granular soils subjected to foundation loads occurs primarily from the compression of the soil skeleton due to rearrangement of particles. The permeability of coarse-grained soil is large enough to justify the assumption of immediate excess pore pressure dissipation upon application of load. Settlement of coarse-grained soil can also be induced by vibratory ground motion due to earthquakes, blasting or machinery, or by soaking and submergence.

3. APPLICABILITY. Settlement estimates discussed in this chapter are applicable to cases where shear stresses are well below the shear strength of the soil.

Section 2. ANALYSIS OF STRESS CONDITIONS

1. MECHANICS OF CONSOLIDATION. See Figure 1. Superimposed loads develop pore pressures in compressible strata exceeding the original hydrostatic pressures. As pore pressure gradients force water from a compressible stratum, its volume decreases, causing settlement.

2. INITIAL STRESSES. See Figure 2 for profiles of vertical stress in a compressible stratum prior to construction. For equilibrium conditions with no excess hydrostatic pressures, compute vertical effective stress as shown in Case 1, Figure 2.



COMPUTATION OF SETTLEMENT

ΔH = SETTLEMENT FROM CONSOLIDATION

Δe = DECREASE IN VOID RATIO CORRESPONDING TO A STRESS INCREASE FROM P_0 TO $(P_0 + \Delta P)$ AT THE MID-HEIGHT OF THE LAYER H_t.

IF Δe IS DETERMINED DIRECTLY ON THE (e-LOGP) CURVE FROM LABORATORY CONSOLIDATION TEST, ΔH IS COMPUTED AS FOLLOWS:

$$\frac{\Delta e}{1+e_0} = \frac{\Delta H}{H_t} \qquad \Delta H = \frac{\Delta e}{1+e_0}(H_t)$$

IF COMPRESSION INDEX C_c IS INTERPRETED FROM A SERIES OF SEMILOGARITHMIC (e-P) CURVES OF CONSOLIDATION TESTS, ΔH IS COMPUTED AS FOLLOWS:

$$\Delta H = \frac{C_c H_t}{1+e_0}\left(\log \frac{P_0 + \Delta P}{P_0}\right)$$

ΔH MAY BE COMPUTED FROM a_v THE SLOPE OF ARITHMETIC (e-P) CURVES, IN THE RANGE FROM (P_0) TO $(P_0 + \Delta P)$:

$$\Delta H = \frac{a_v \Delta P H_t}{1+e_0} \qquad a_v = \frac{0.435\, C_c}{P_0 + \Delta P/2}$$

COMPUTATION OF TIME RATE OF CONSOLIDATION

$\bar{U}$ = AVERAGE PERCENT OF CONSOLIDATION COMPLETED AT ANY TIME t AND DEPENDS ON THE DEGREE OF DISSIPATION OF THE INITIAL HYDROSTATIC EXCESS PORE WATER PRESSURES u_0.

$\bar{U}$ AT ANY TIME IS MEASURED BY THE DIVISION OF THE AREA UNDER THE INITIAL EXCESS PRESSURE DIAGRAM BETWEEN EFFECTIVE STRESS AND PORE PRESSURE:

$$\bar{U} = \frac{A_\sigma}{u_0 H_t} = \frac{A_\sigma}{A_\sigma + A_u}$$

THIS RELATIONSHIP IS EVALUATED BY THE THEORY OF CONSOLIDATION AND IS EXPRESSED BY THE TIME FACTOR T_v. TO DETERMINE $\bar{U}$ AS A FUNCTION OF TIME FACTOR, USE CURVES OF FIGURE 9.

$$T_v = \frac{C_v t}{H^2}$$

H = LENGTH OF LONGEST VERTICAL PATH FOR DRAINAGE OF PORE WATER. FOR DRAINAGE TO PERVIOUS LAYERS AT TOP AND BOTTOM OF COMPRESSIBLE STRATUM, $H = H_t/2$.

FIGURE 1
Consolidation Settlement Analysis

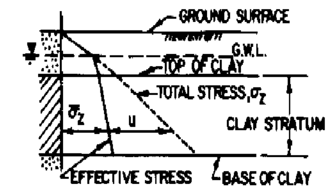
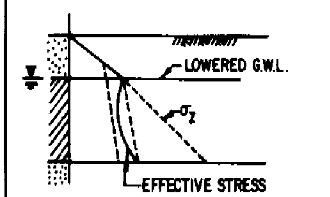
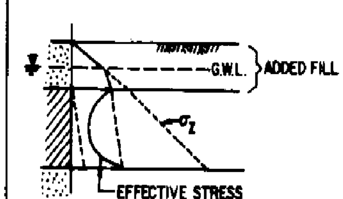
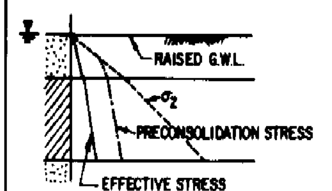
	STRESS CONDITION	DIAGRAM OF VERTICAL STRESSES	DESCRIPTION
	(1) SIMPLE OVERBURDEN PRESSURE		TOTAL STRESS σ_z IS COMPUTED USING TOTAL UNIT WEIGHT γ_T BOTH ABOVE AND BELOW THE G.W.L. PORE WATER PRESSURE u IS DUE TO G.W.L. EFFECTIVE STRESS $\bar{\sigma}_z = \sigma_z - U$.
HYDROSTATIC EXCESS PRESSURE	(2) LOWERING OF GROUND WATER LEVEL		IMMEDIATELY AFTER LOWERING OF THE GROUNDWATER TOTAL STRESS IN TOP SAND LAYER REMAINS PRACTICALLY UNCHANGED, BUT THE EFFECTIVE STRESSES INCREASE. SINCE THE WATER ESCAPES SLOWLY FROM THE CLAY LAYER, THE EFFECTIVE STRESS REQUIRES LONG TIME TO REACH THE NEW EQUILIBRIUM VALUE.
	(3) PARTIAL CONSOLIDATION UNDER WEIGHT OF INITIAL FILL		TOTAL STRESSES ON A CLAY LAYER INCREASED BY THE ADDITION OF SURCHARGE LOAD. INITIALLY THIS LOAD IS CARRIED BY PORE WATER IN THE FORM OF EXCESS PORE PRESSURE. AS THE SETTLEMENT PROGRESSES IN THE CLAY LAYER, THE EFFECTIVE STRESS INCREASES TO CORRESPOND TO THE STRESS FROM SURCHARGE LOAD.
PRECONSOLIDATED CONDITIONS	(4) RISE OF GROUND WATER LEVEL		RISE OF GROUND WATER LEVEL DECREASES EFFECTIVE PRESSURE OF OVERBURDEN. EFFECTIVE STRESS LINE MOVES TO LEFT. THEN PRECONSOLIDATION STRESS EQUALS ORIGINAL EFFECTIVE STRESS OVERBURDEN. TOTAL STRESS PRACTICALLY UNCHANGED.

FIGURE 2
Profiles of Vertical Stresses Before Construction

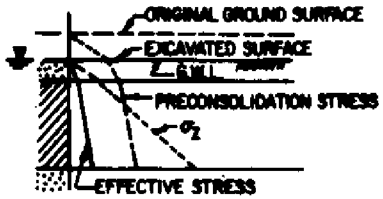
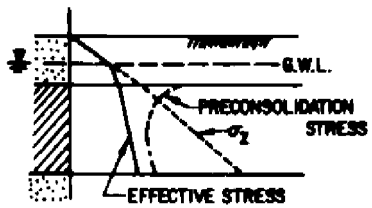
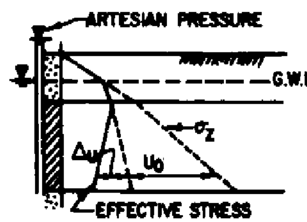
<table>
<tr><th></th><th>STRESS CONDITION</th><th>DIAGRAM OF VERTICAL STRESSES</th><th>DESCRIPTION</th></tr>
<tr><td rowspan="2">PRECONSOLIDATED CONDITIONS</td><td>(5)
EXCAVATION</td><td>
</td><td>EXCAVATION OF OVERBURDEN MATERIAL UNLOADS CLAY LAYER. EFFECTIVE STRESS LINE MOVES TO THE LEFT. THEN PRECONSOLIDATION STRESS EQUALS ORIGINAL EFFECTIVE STRESS OF OVERBURDEN.</td></tr>
<tr><td>(6)
PRECONSOLIDATION FROM LOADING IN THE PAST</td><td>
</td><td>PRECONSOLIDATION FROM PAST LOADINGS GREATER THAN THE EXISTING OVERBURDEN MAY HAVE BEEN CAUSED BY WEIGHT OF GLACIAL ICE, EROSION OF FORMER OVERBURDEN, LOWER GROUND WATER LEVEL PLUS DESSICATION, OR REMOVAL OF FORMER STRUCTURES.</td></tr>
<tr><td colspan="2">(7)
ARTESIAN PRESSURE</td><td>
</td><td>SAND STRATUM BELOW THE CLAY MAY BE SUBJECT TO ARTESIAN HYDRAULIC PRESSURES THAT DECREASE EFFECTIVE STRESS AT BASE OF CLAY. TOTAL STRESS REMAINS UNCHANGED.</td></tr>
</table>

FIGURE 2 (continued)
Profiles of Vertical Stresses Before Construction

a. Preconsolidation. Stresses exceeding the present effective vertical pressure of overburden produce preconsolidation (1) by the weight of material that existed above the present ground surface and that has been removed by erosion, excavation, or recession of glaciers, (2) by capillary stresses from desiccation, and (3) by lower groundwater levels at some time in the past.

b. Underconsolidation. Compressible strata may be incompletely consolidated under existing loads as a result of recent lowering of groundwater or recent addition of fills or structural loads. Residual hydrostatic excess pore pressure existing in the compressible stratum will dissipate with time, causing settlements.

c. Evaluation of Existing Conditions. Determine consolidation condition at start of construction by the following steps:

(1) Review the data available on site history and geology to estimate probable preconsolidation or underconsolidation.

(2) Compare profile of preconsolidation stress determined from laboratory consolidation tests (Chapter 3) with the profile of effective over-burden pressures.

(3) Estimate preconsolidation from c/P+c, ratio, where c is the cohesion (q+u/2,) and P+c, is the preconsolidation stress, using laboratory data from unconfined compression test and Atterberg limits (see Chapter 3).

(4) If underconsolidation is indicated, install piezometers to measure the magnitude of hydrostatic excess pore water pressures.

d. Computation of Added Stresses. Use the elastic solutions (Chapter 4) to determine the vertical stress increment from applied loads. On vertical lines beneath selected points in the loaded area, plot profiles of estimated preconsolidation and effective overburden stress plus the increment of applied stress. See Figure 3 for typical profiles. Lowering of groundwater during construction or regional drawdown increases effective stress at the boundaries of the compressible stratum and initiates consolidation. Stress applied by drawdown equals the reduction in buoyancy of overburden corresponding to decrease in boundary water pressure. In developed locations, settlement of surrounding areas from drawdown must be carefully evaluated before undertaking dewatering or well pumping.

Section 3. INSTANTANEOUS SETTLEMENT

1. IMMEDIATE SETTLEMENT OF FINE-GRAINED SOILS. Generally, the instantaneous settlement results from elastic compression of clayey soil. For foundations on unsaturated clay or highly overconsolidated clay, the elastic settlement constitutes a significant portion of the total settlement.

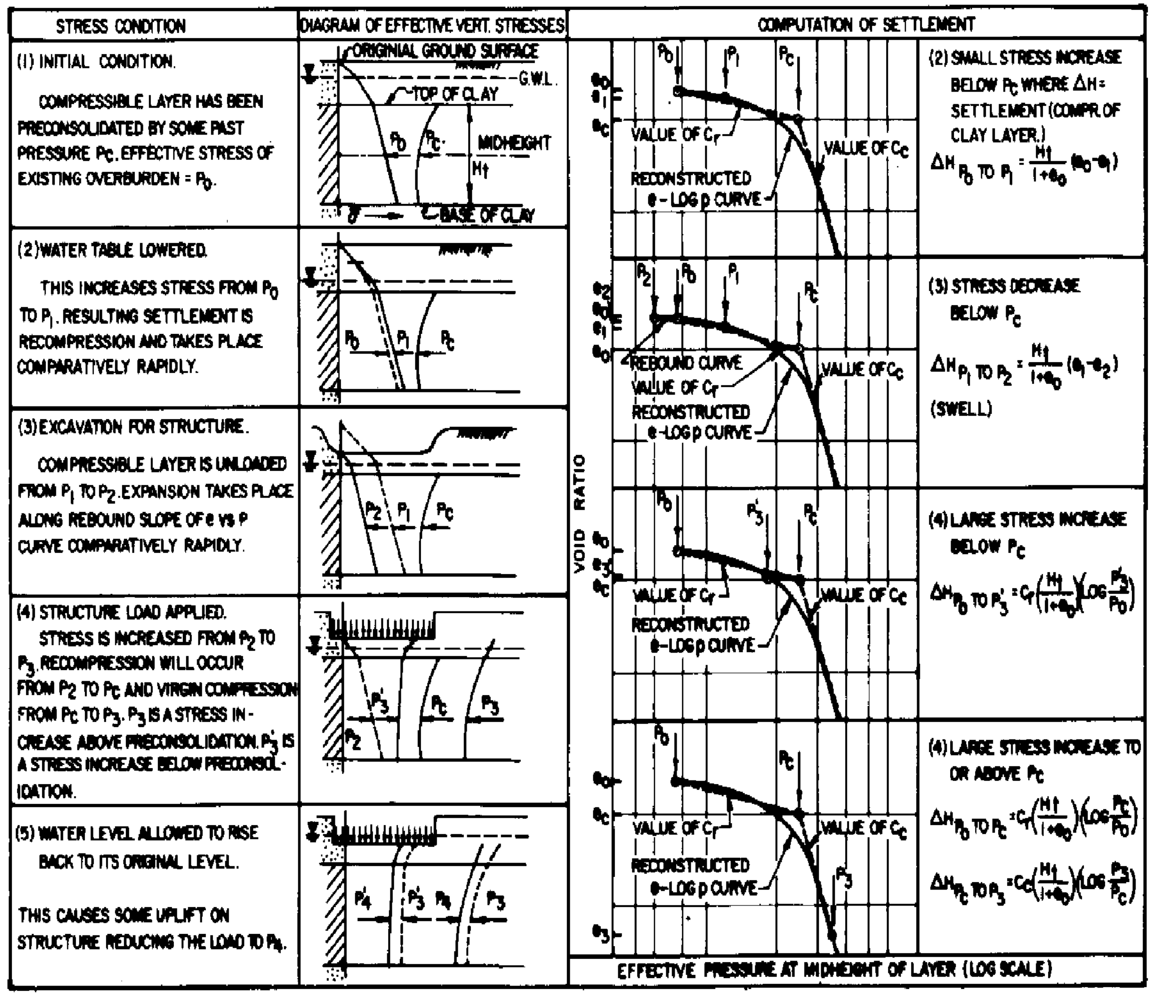


FIGURE 3
Computation of Total Settlement for Various Loading Conditions

Immediate settlement δ_v, is estimated as:

$$\delta_v = q\,B \left[\frac{1-\gamma^2}{E_u} \right] I$$

q is applied uniform pressure; B is width of loaded area; I is combined shape and rigidity factor; γ is Poisson's ratio - ranges between 0.3 and 0.5, the higher value being for saturated soil with no volume change during loading; and E_u is undrained modulus obtained from laboratory or field (pressuremeter) tests. Table 1 (Reference 1, <u>Stresses and Deflections in Foundations and Pavements</u>, by Department of Civil Engineering, University of California, Berkeley) provides values of I. Empirical relationship derived from field measurement may be used to determine E_u when actual test values are not available; see Table 2 (adapted from Reference 2, <u>An Engineering Manual For Settlement Studies</u>, by Duncan and Buchignani). Empirical correlations for estimation of OCR (Over Consolidation Ratio) are presented in Chapter 3.

If the factor of safety against bearing failure (see DM-7.2, Chapter 4) is less than about 3, then the immediate settlement δ_v is modified as follows:

$$\delta_c = \delta_v / SR$$

δ_c = immediate settlement corrected to allow for partial yield condition

SR = Settlement Ratio

Determine SR from Figure 4 (Reference 3, <u>Initial Settlement of Structures on Clay</u>, by D'Appolonia, et al.). See Figure 5 for an example.

2. SETTLEMENT OF COARSE-GRAINED SOILS. This immediate settlement is a function of the width and depth of footing, elevation of the water table, and the modulus of vertical subgrade reaction (K_{VI}) within the depth affected by the footing. Figure 6 may be used to estimate K_{VI} from the soil boring log, and to compute anticipated settlement.

For large footings where soil deformation properties vary significantly with depth or where the thickness of granular soil is only a fraction of the width of the loaded area, the method in Figure 6 may underestimate settlement.

3. TOTAL SETTLEMENT IN GRANULAR SOILS. Total settlement is the combined effect of immediate and long-term settlements. A usually conservative estimate of settlement can be made utilizing the method in Figure 7 (Reference 4, <u>Static Cone to Compute Static Settlement Over Sand</u>, by Schmertmann). A review of methods dealing with settlement of sands utilizing the standard penetration test results can be found in Reference 5, <u>Equivalent Linear Model for Predicting Settlements of Sand Bases</u>, by Oweis.

TABLE 1
Shape and Rigidity Factors I for Calculating Settlements
of Points on Loaded Areas at the Surface of an Elastic Half-Space

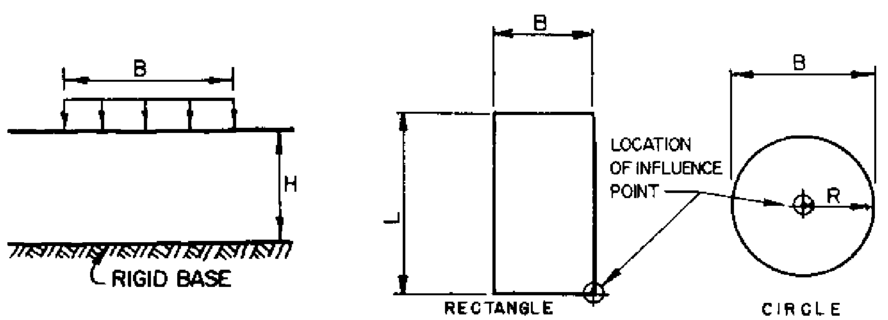
Shape and Rigidity Factor I for Loaded Areas
on an Elastic Half-Space of Infinite Depth

Shape and Rigidity	Center	Corner	Edge/Middle of Long Side	Average
Circle (flexible)	1.00		0.64	0.85
Circle (rigid)	0.79		0.79	0.79
Square (flexible)	1.12	0.56	0.76	0.95
Square (rigid)	0.82	0.82	0.82	0.82
Rectangle:				
(flexible)				
length/width				
2	1.53	0.76	1.12	1.30
5	2.10	1.05	1.68	1.82
10	2.56	1.28	2.10	2.24
Rectangle:				
(rigid)				
length/width				
2	1.12	1.12	1.12	1.12
5	1.6	1.6	1.6	1.6
10	2.0	2.0	2.0	2.0

TABLE 1 (continued)
Shape and Rigidity Factors I for Calculating Settlements
of Points on Loaded Areas at the Surface of an Elastic Half-Space

Shape and Rigidity Factor I for Loaded Areas
on an Elastic Half-Space of Limited Depth Over a Rigid Base

H/B	Center of Rigid Circular Area Diameter = B	Corner of Flexible Rectangular Area				
		L/B = 1	L/B = 2	L/B = 5	L/B = 10	(strip) L/B = ∞
for ν = 0.50						
0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.14	0.05	0.04	0.04	0.04	0.04
1.0	0.35	0.15	0.12	0.10	0.10	0.10
1.5	0.48	0.23	0.22	0.18	0.18	0.18
2.0	0.54	0.29	0.29	0.27	0.26	0.26
3.0	0.62	0.36	0.40	0.39	0.38	0.37
5.0	0.69	0.44	0.52	0.55	0.54	0.52
10.0	0.74	0.48	0.64	0.76	0.77	0.73
for ν = 0.33						
0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.20	0.09	0.08	0.08	0.08	0.08
1.0	0.40	0.19	0.18	0.16	0.16	0.16
1.5	0.51	0.27	0.28	0.25	0.25	0.25
2.0	0.57	0.32	0.34	0.34	0.34	0.34
3.0	0.64	0.38	0.44	0.46	0.45	0.45
5.0	0.70	0.46	0.56	0.60	0.61	0.61
10.0	0.74	0.49	0.66	0.80	0.82	0.81

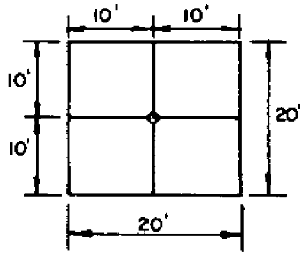


NOTATION FOR LOADED AREAS, SHOWN IN PLAN VIEW

TABLE 1 (continued)
Shape and Rigidity Factors I for Calculating Settlements
of Points on Loaded Areas at the Surface of an Elastic Half-Space

Example:

Compute immediate settlement at center of uniformly loaded area (flexible) measuring 20' by 20'.



Calculate as the sum of the influence values at the corners of four equal-sided rectangles.

$$\delta_v = qB \frac{1-\nu^2}{E_u} I$$

$q = 4$ KSF, $B = 10'$

$\nu = 0.5$, $E_u = 20$ KSF

$H/B = 1$, $L/B = 1$ $I = 0.15$

$$\delta = 4 \times 10 \times \left[\frac{1-0.5^2}{20}\right] \times 0.15$$

$$= 0.225'$$

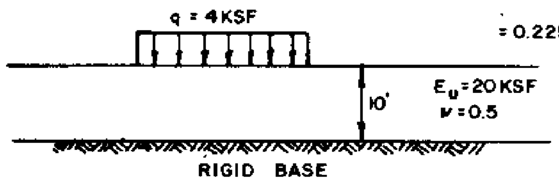


TABLE 2
Relationship Between Undrained Modulus and Overconsolidation Ratio

OCR[*]	Eu/c		
	PI<30	30<PI<50	PI>50
<3	600	300	125
3 - 5	400	200	75
>5	150	75	50

[*] OCR = Overconsolidation ratio

c = Undrained shear strength

PI = Plastic index

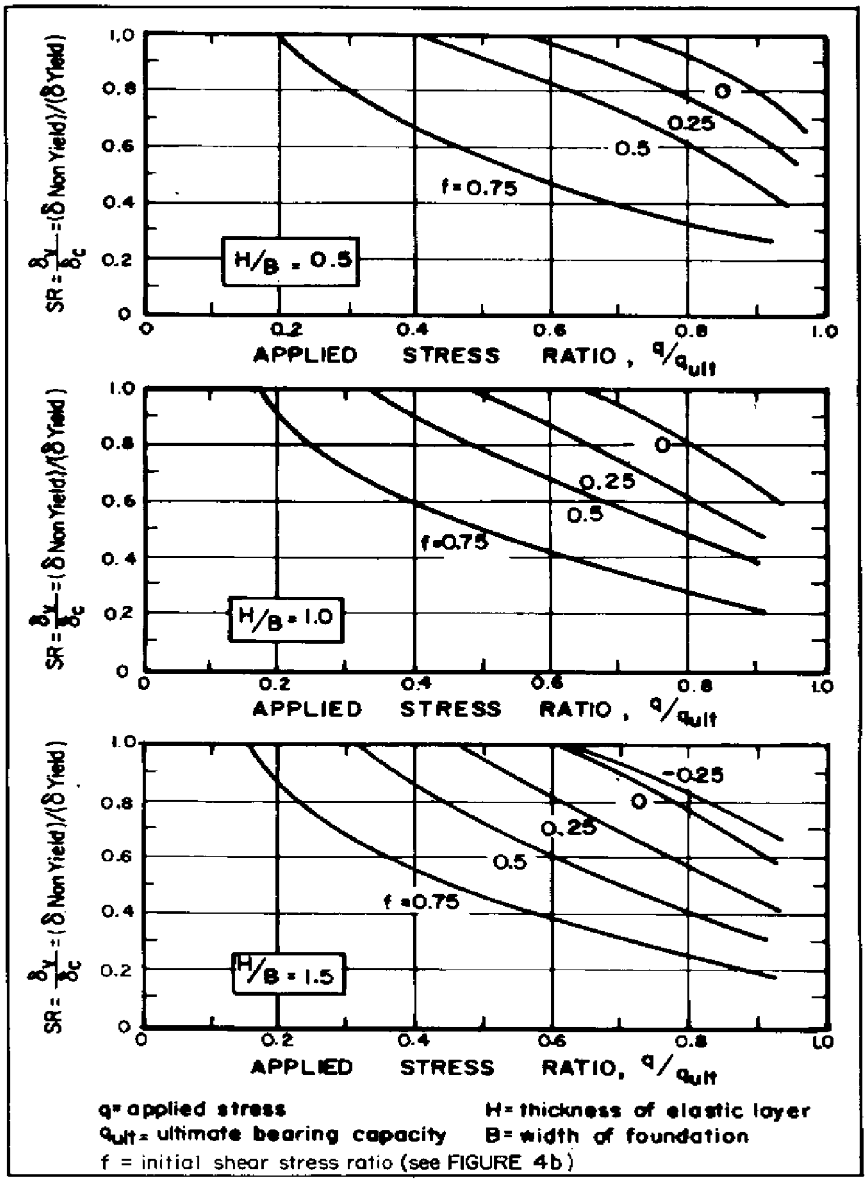


FIGURE 4a
Relationship Between Settlement Ratio and Applied Stress Ratio for Strip Foundation on Homogeneous Isotropic Layer

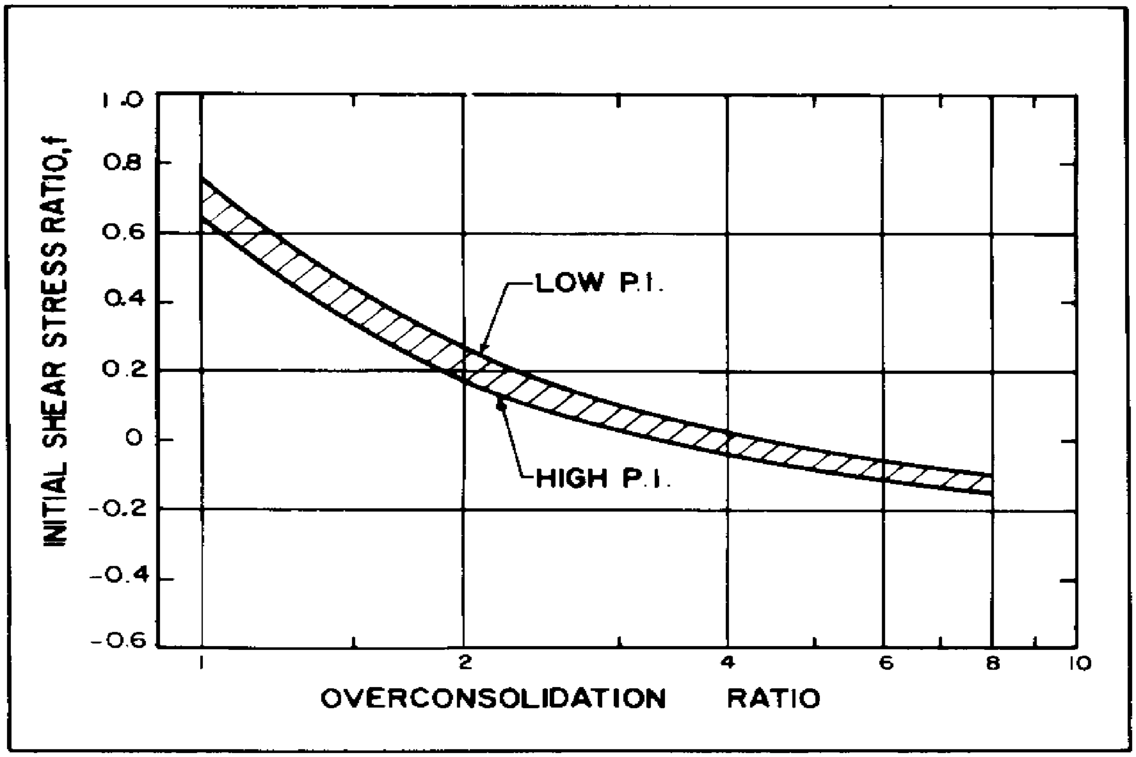


FIGURE 4b
Relationship Between Initial Shear Stress and Overconsolidation Ratio

Example:

Given LL = 58%   PI = 25%   c = 1 KSF

Moderately consolidated clay, OCR <3

Depth to rigid layer (H) = 10.5 ft

$\gamma = 0.5$

Rigid strip footing, width = 7 ft $q_{appl} = 2.5$ KSF $q_{ult} = 6$ KSF

Find immediate settlement.

$$\delta_v = qB \frac{(1-\gamma^2)}{E_u} I$$

$I = 2.0$ (Table 1) assume length/width ≈ 10

From Table 2, $E_u/c = 600$

$E_u = 600 \times 1 = 600$ KSF

$$\delta_v = \frac{2.5 \times 7 \times (1-0.5^2) \times 2.0}{600} \times 12 = 0.52 \text{ inches}$$

Find factor of safety against bearing failure.

$$F_S = \frac{6.0}{2.5} = 2.4,\ 2.4 < 3.0$$

Correct for yield.

$f = 0.7$ (Figure 4b)

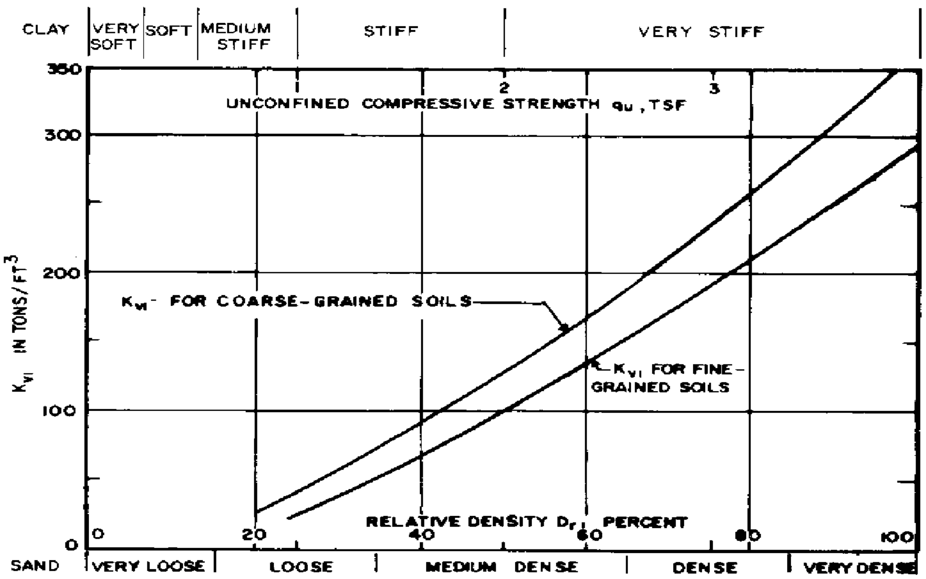
$q_{appl}/q_{ult} = 0.42$, $H/B = 1.5$

SR = 0.60 (Figure 4a)

Corrected value of initial settlement

$$\delta_c = \frac{0.52}{0.60} = 0.87 \text{ inches}$$

FIGURE 5
Example of Immediate Settlement Computations in Clay



DEFINITIONS

ΔH_i = IMMEDIATE SETTLEMENT OF FOOTING
q = FOOTING UNIT LOAD IN tsf
B = FOOTING WIDTH
D = DEPTH OF FOOTING BELOW GROUND SURFACE
K_{vi} = MODULUS OF VERTICAL SUBGRADE REACTION

COARSE-GRAINED SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leq B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{4\, q\, B^2}{K_{vi}\,(B+1)^2}$$

FOR $B \geq 40$ FT:

$$\Delta H_i = \frac{2\, q\, B^2}{K_{vi}\,(B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geq 5B$

FOR $B \leq 20$ FT:

$$\Delta H_i = \frac{2\, q\, B^2}{K_{vi}\,(B+1)^2}$$

NOTES: 1. NONPLASTIC SILT IS ANALYZED AS COARSE-GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.

2. VALUES OF K_{vi} SHOWN FOR COARSE-GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUNDWATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUNDWATER IS AT BASE OF FOOTING, USE $K_{vi}/2$ IN COMPUTING SETTLEMENT

3. FOR CONTINUOUS FOOTINGS MULTIPLY THE SETTLEMENT COMPUTED FOR WIDTH "B" BY 2.

FIGURE 6
Instantaneous Settlement of Isolated Footings on Coarse-Grained Soils

DATA REQUIRED:

1. A profile of standard penetration resistance N (blows/ft) versus depth, from the proposed foundation level to a depth of 2B, or to boundary of an incompressible layer, whichever occurs first. Value of soil modulus E_s is established using the following relationships.

Soil Type	E_s/N
Silts, sands silts, slightly cohesive silt-sand mixtures	4
Clean, fine to med, sands & slightly silty sands	7
Coarse sands & sands with little gravel	10
Sandy gravels and gravel	12

2. Least width of foundation = B, depth of embedment = D, and proposedaverage contact pressure = P.

3. Approximate unit weights of surcharge soils, and position of water table if within D.

4. If the static cone bearing value q_c measured compute E_s based on $E_s = 2\ q_c$.

ANALYSIS PROCEDURE:

Refer to table in example problem for column numbers referred to by parenthesis:

1. Divide the subsurface soil profile into a convenient number of layers of any thickness, each with constant N over the depth interval 0 to 2B below the foundation.

2. Prepare a table as illustrated in the example problem, using the indicated column headings. Fill in columns 1, 2, 3 and 4 with the layering assigned in Step 1.

3. Multiply N values in column 3 by the appropriate factor E_s/N (col. 4) to obtain values of E_s; place values in column 5.

4. Draw an assumed 2B-0.6 triangular distribution for the strain influence factor I_z, along a scaled depth of 0 to 2B below the foundation. Locate the depth of the mid-height of each of the layers assumed in Step 2, and place in column 6. From this construction, determine the I_z value at the mid-height of each layer, and place in column 7.

FIGURE 7
Settlement of Footings Over Granular Soils: Example Computation Using Schmertmann's Method

5. Calculate $(I_z/E_s)\ \Delta Z$, and place in column 8. Determine the sum of all values in column 8.

6. Total settlement = $\Delta H = C_1 C_2 \Delta p \sum_0^{2B} \left(\frac{I_z}{E_s}\right) \Delta Z$,

where $C_1 = 1 - 0.5\ (p_o/\Delta p)$; $C_1 \geq 0.5$ embedment correction factor

$C_2 = 1 + 0.2 \log (10t)$ creep correction factor

p_o = overburden pressure at foundation level

Δp = net foundation pressure increase

t = elaspsed time in years.

EXAMPLE PROBLEM:

GIVEN THE FOLLOWING SOIL SYSTEM AND CORRESPONDING STANDARD PENETRATION TEST (SPT) DATA, DETERMINE THE AMOUNT OF ULTIMATE SETTLEMENT UNDER A GIVEN FOOTING AND FOOTING LOAD:

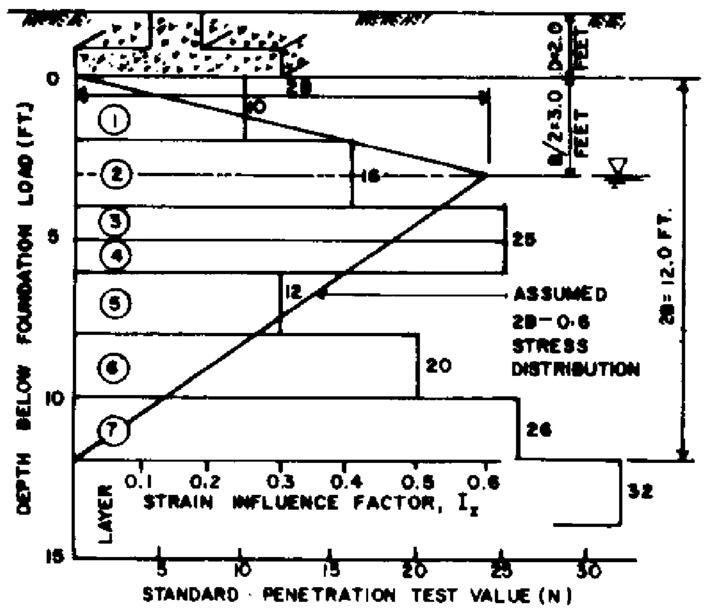


FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Footing Details:

Footing width: 6.0 ft. (min.) by 8.0 ft. (max.)

Depth of Embedment: 2.0 ft. Load (Dead + Live): 120 tons

Soil Properties:

Depth Below Surface (ft.)	Depth Below Base of Footing (ft.)	Unit Wt. (pcf) Moist	Sat.	Soil Description
0 - 5	<5	95	105	Fine sandy silt
5 - 10	3 - 8	105	120	Fine to medium sand
10 - 17	8 - 15	120	130	Coarse sand

Solution:

Layer (1)	ΔZ (in.) (2)	N (3)	E_s/N (4)	E_s (tsf) (5)	Z_c (in.) (6)	I_z (7)	$\frac{I_z}{E_s}\Delta Z$ (in./tsf) (8)
1	24	10	4	40	12	.20	0.120
2	24	16	4	64	36	.60	0.225
3	12	25	4	100	54	.50	0.060
4	12	25	7	175	66	.43	0.029
5	24	12	7	84	84	.33	0.094
6	24	20	7	140	108	.20	0.034
7	24	26	10	260	132	.07	0.006

$\Sigma = 0.568$

$p_o = (2.0 \text{ ft})(95 \text{ pcf}) = 190 \text{ psf} = 0.095 \text{ tsf}$

$\Delta p = 120 \text{ tons}/(6 \text{ ft.})(8 \text{ ft.}) = 2.50 \text{ tsf}$

At t = 1 yr,

$$C_1 = 1 - 0.5(.095/2.50) = 0.981$$

$$C_2 = 1 + 0.2 \log (10)(1) = 1.20$$

$$\Delta H = (0.981)(1.20)(2.50)(0.568) = \underline{1.67 \text{ in.}}$$

FIGURE 7 (continued)
Settlement of Footings Over Granular Soils:
Example Computation Using Schmertmann's Method

Section 4. PRIMARY AND SECONDARY SETTLEMENTS

1. PRIMARY CONSOLIDATION.

a. Consolidation Settlement. For conditions where excess pore pressures are developed during the application of load and if preconsolidation stress is determined reliably, total settlement can be predicted with reasonable accuracy. The percentage error is greatest for settlement from recompression only. In this case an overestimate may result unless high quality undisturbed samples are used for consolidation tests.

(1) Typical Loading Cycle. See Figure 3 for loading sequence in building construction. Foundation excavation can cause swell and heave. Application of a structural load recompresses subsoil and may extend consolidation into the virgin compression range. Stress changes are plotted on a semilogarithmic pressure-void ratio e-log p curve similar to that shown in Figure 3.

(2) Pressure-Void Ratio Diagram. Determine the appropriate e-log p curve to represent average properties of compressible stratum from consolidation tests. The e-log p curve may be interpreted from straight line virgin compression and recompression slopes intersecting at the preconsolidation stress. Draw e-log p curve to conform to these straight lines as shown in Figure 3.

(3) Magnitude of Consolidation Settlement. Compute settlement magnitude from change in void ratio corresponding to change in stress from initial to final conditions, obtained from the e-log p curve (Figure 3). To improve the accuracy of computations divide the clay layer into a number of sublayers for computing settlement. Changes in compressibility of the stratum and existing and applied stresses can be dealt with more accurately by considering each sublayer independently and then finding their combined effect.

(4) Preliminary estimates of C+c, can be made using the correlations in Table 3.

b. Corrections to Magnitude of Consolidation Settlements. Settlements computed for overconsolidated clays by the above procedures may give an overestimate of the settlement. Correct consolidation settlement estimate as follows:

H+c, =[alpha] ([W-DELTA]H)+oc,

H+c, = corrected consolidation settlement

[alpha] = function of overconsolidation ratio (OCR) and the width of loaded area and thickness of compressible stratum (See Figure 8 for values and Reference 6, Estimating Consolidation Settlements of Shallow Foundation on Overconsolidated Clay, by Leonards.)

TABLE 3
Estimates of Coefficient of Consolidation (C_c)

$C_c = 0.009\ (LL - 10\%)$ inorganic soils, with sensitivity less than 4

$C_c = 0.0115\ w_n$, organic soils, peat

$C_c = 1.15\ (e_o - 0.35)$ all clays

$C_c = (1 + e_o)(0.1 + [w_n - 25]\ 0.006)$ varved clays

w_n is natural moisture content, LL is water content at liquid limit and e_o is initial void ratio.

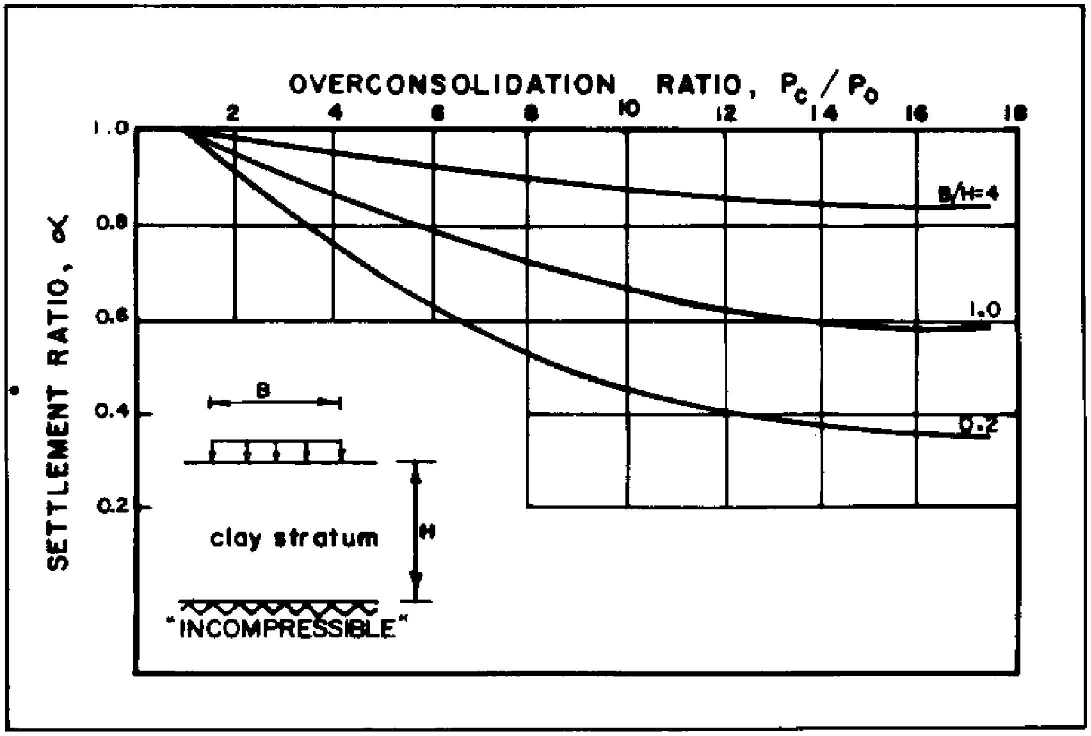


FIGURE 8
Relation Between Settlement Ratio and Overconsolidation Ratio

OCR = preconsolidation pressure/overburden pressure (P_c/P_o) (See Chapter 3.)

$(\Delta H)_{oc}$ = calculated settlement resulting from stress increment of P_o to P_c by procedures outlined in Figure 3, Section 2.

2. TIME RATE OF PRIMARY CONSOLIDATION.

a. Application. Settlement time rate must be determined for foundation treatment involving either acceleration of consolidation or preconsolidation before construction of structure. Knowledge of settlement rate or percent consolidation completed at a particular time is important in planning remedial measures on a structure damaged by settlement.

b. Time Rate of Consolidation. Where pore water drainage is essentially vertical, the ordinary one dimensional theory of consolidation defines the time rate of settlement. Using the coefficient of consolidation c_v, compute percent consolidation completed at specific elapsed times by the time factor T_v, curves of Figure 9 (upper panel, Reference 7, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), by the Departments of the Army and Air Force). For vertical sand drains use Figure 10 (upper panel, Reference 7). For preliminary estimates, the empirical correlation for c_v in Chapter 3 may be used.

(1) Effect of Pressure Distribution. Rate of consolidation is influenced by the distribution of the pressures which occur throughout the depth of the compressible layer. For cases where the pressures are uniform or vary linearly with depth, use Figure 9 which includes the most common pressure distribution. The nomograph in Figure 11 may be used for this case.

For nonlinear pressure distribution, use Reference 8, Soil Mechanics in Engineering Practice, by Terzaghi and Peck, to obtain the time factor.

(2) Accuracy of Prediction. Frequently the predicted settlement time is longer than that observed in the field for the following reasons:

(a) Theoretical conditions assumed for the consolidation analysis frequently do not hold in situ because of intermediate lateral drainage, anisotropy in permeability, time dependency of real loading, and the variation of soil properties with effective stress. Two or three dimensional loading increases the time rate of consolidation. Figure 12 (after Reference 9, Stress Deformation and Strength Characteristics, by Ladd et al.) gives examples of how the width of the loaded area and anistropy in permeability can affect the consolidation rate substantially. As the ratio of the thickness of the compressible layer to the width of the loaded area increases, the theory tends to overestimate the time factor. For deposits such as some horizontal varved clays where continuous seams of high permeability are present, consolidation can be expected to be considerably faster than settlement rates computed based on the assumption of no lateral drainage.

(b) The coefficient of consolidation, as determined in the laboratory, decreases with sample disturbance. Predicted settlement time tends to be greater than actual time (see Chapter 3).

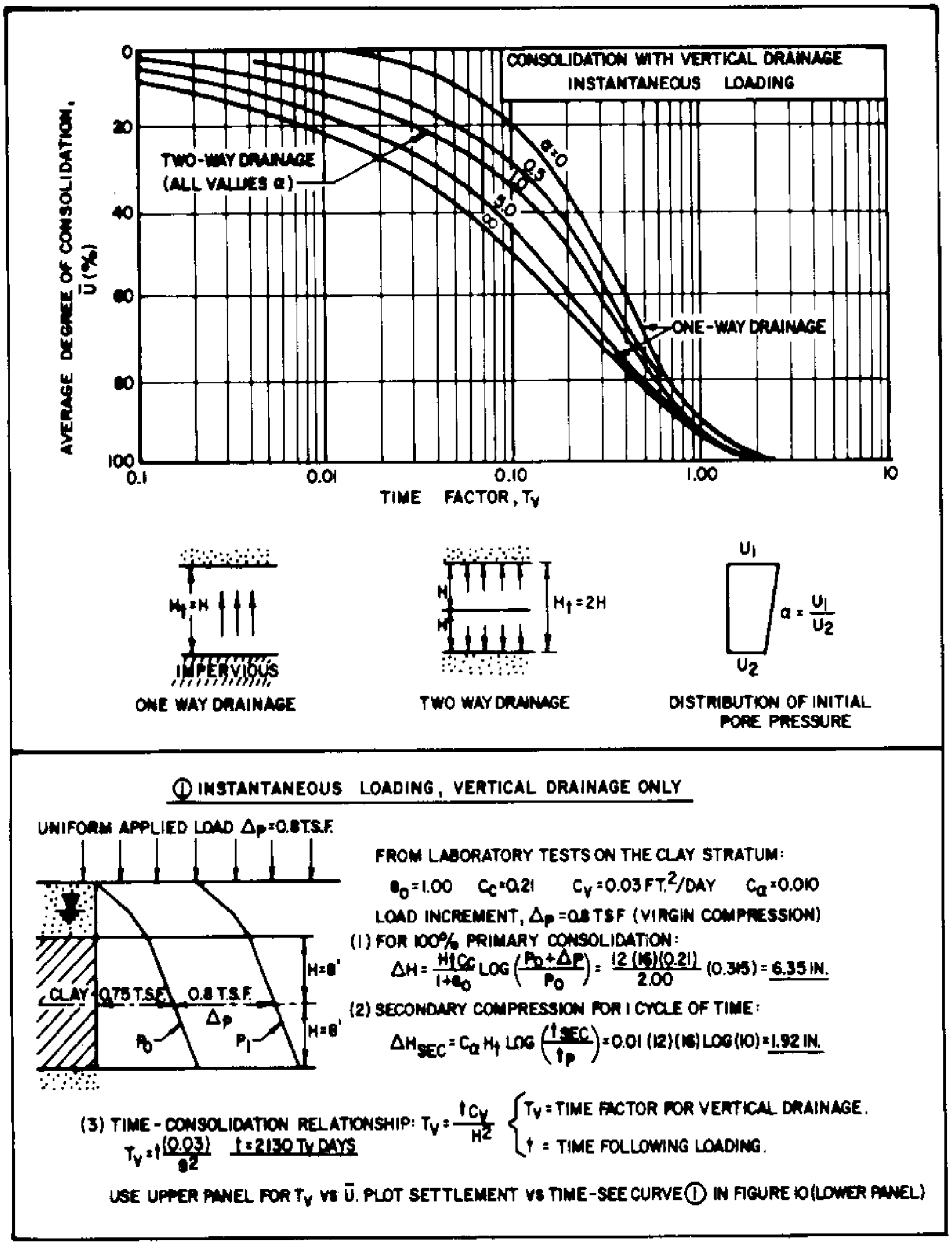
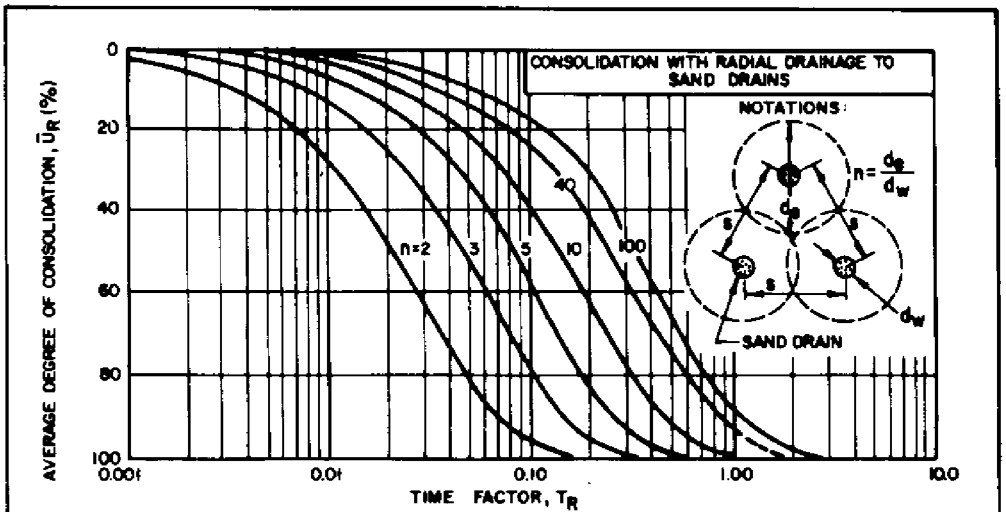
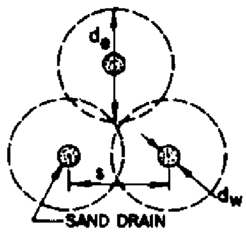


FIGURE 9
Time Rate of Consolidation for Vertical Drainage
Due to Instantaneous Loading





③ INSTANTANEOUS LOADING WITH SAND DRAINS

ANALYZE SAND DRAINS FOR ACCELERATION OF CONSOLIDATION.
DRAINS IN TRIANGULAR ARRAY SPACING, S=10' d_e=10.5', WELL DIAMETER, d_w=18"

HORIZONTAL COEFFICIENT OF CONSOLIDATION, $c_h = 0.06\ FT^2/DAY$

$$T_R = \frac{t\ c_h}{(d_e)^2},\quad t = 1840\ T_R \quad n = \frac{d_e}{d_w} = \frac{10.5}{1.5} = 7$$

USE CURVE IN GRAPH ABOVE FOR n=7 TO OBTAIN T_R VS $\bar{U}_R$. PLOT (t VS $\bar{U}_R$) AS CURVE ②. BELOW COMBINED EFFECT OF VERTICAL AND RADIAL DRAINAGE:

$$\bar{U}_c = 100 - \left[\frac{(100-\bar{U}_R)(100-\bar{U}_V)}{100}\right].$$ PLOT (t VS $\bar{U}_c$) AS CURVE ③ BELOW.

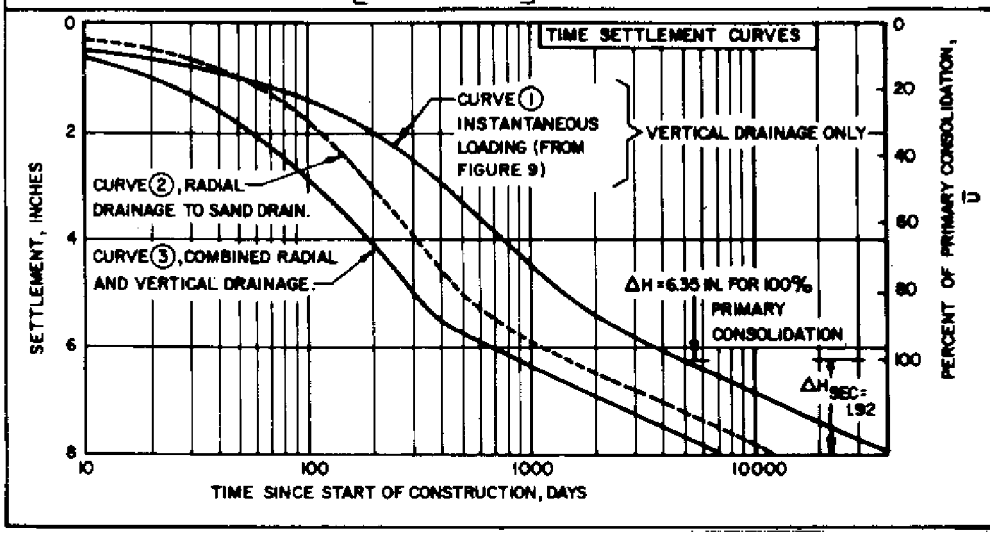


FIGURE 10
Vertical Sand Drains and Settlement Time Rate
7.1-228

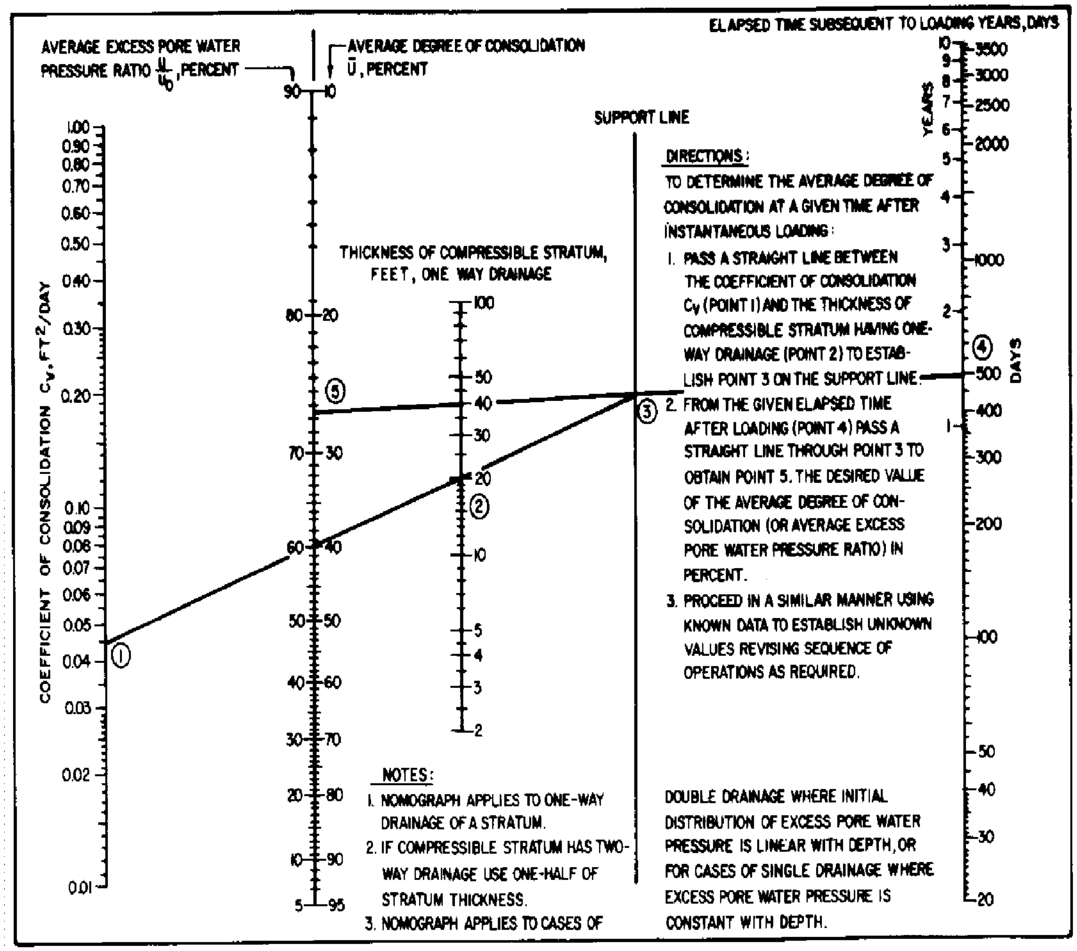
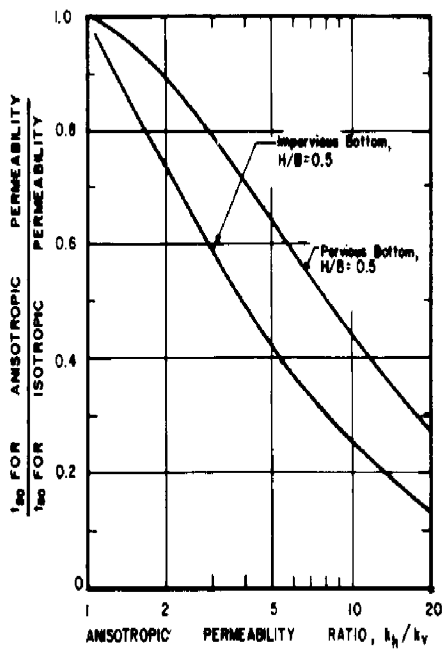
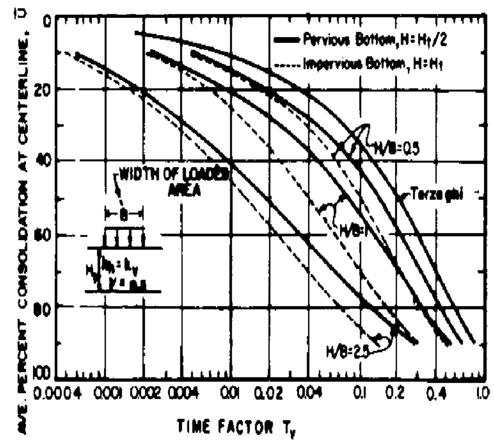


FIGURE 11
Nomograph for Consolidation With Vertical Drainage





Find time needed for 50% consolidation.

c_v = 0.03 ft^2/day   H/B=2.5 k_h/k_v=5

H_t=16 ft., drainage at top and bottom

Drainage distance,H=8 ft.

T_v = 0.02

$$t_{50} \text{ (for isotropic permeability)} = \frac{0.02 \times 8^2}{0.03} = 42.67 \text{ days}$$

Corrected t_{50} for anisotropy.

t_{50} (aniso)/t_{50} (iso)=0.65

Therefore, t_{50} (aniso)=0.65 x 42.67=27.7 days

FIGURE 12

Effect of Drainage Conditions on Time Rate of Consolidation

(3) Gradual Load Application. If construction time is appreciable compared to time required for primary consolidation, use the time factors of Figure 13 (Reference 10, Consolidation Under Time Dependent Loading, by Olson) to determine consolidation rate during and following construction.

(4) Coefficient of Consolidation From Field Measurements. Where piezometers are installed to measure pore water pressure under the applied loads, c_v, is computed as shown in Figure 14.

c. Time Rate of Multi-Layer Consolidation. If a compressible stratum contains layers of different overall properties, use the procedure of Figure 15 to determine overall settlement time rate.

3. SECONDARY COMPRESSION.

a. Laboratory e-log p Curve. A laboratory e-log p curve includes an amount of secondary compression that depends on duration of test loads. Secondary compression continues exponentially with time without definite termination. Thus, total or ultimate settlement includes secondary compression to a specific time following completion of primary consolidation.

b. Settlement Computation. Compute settlement from secondary compression following primary consolidation as follows:

$$H_{sec} = C_{\alpha} (H_t) \log \frac{t_{sec}}{t_p}$$

H_{sec} = settlement from secondary compression

C_{α} = coefficient of secondary compression expressed by the strain per log cycle of time (See Chapter 3)

H_t = thickness of the compressible stratum

t_{sec} = useful life of structure or time for which settlement is significant

t_p = time of completion of primary consolidation

See example in Figure 9 for calculating the secondary settlement. The parameter C can be determined from laboratory consolidation tests (Chapter 3); for preliminary estimates, the correlations in Figure 16 (after Reference 2) may be used. This relationship is applicable to a wide range of soils such as inorganic plastic clays, organic silts, peats, etc.

c. Combining Secondary and Primary Consolidation. If secondary compression is important, compute the settlement from primary consolidation separately, using an e-log p curve that includes only compression from primary consolidation. For each load increment in the consolidation test, compression is plotted versus time (log scale) (see Chapter 3). The compression at the end of the primary portion (rather than standard 24 hours) may be used to establish e-log p curve.

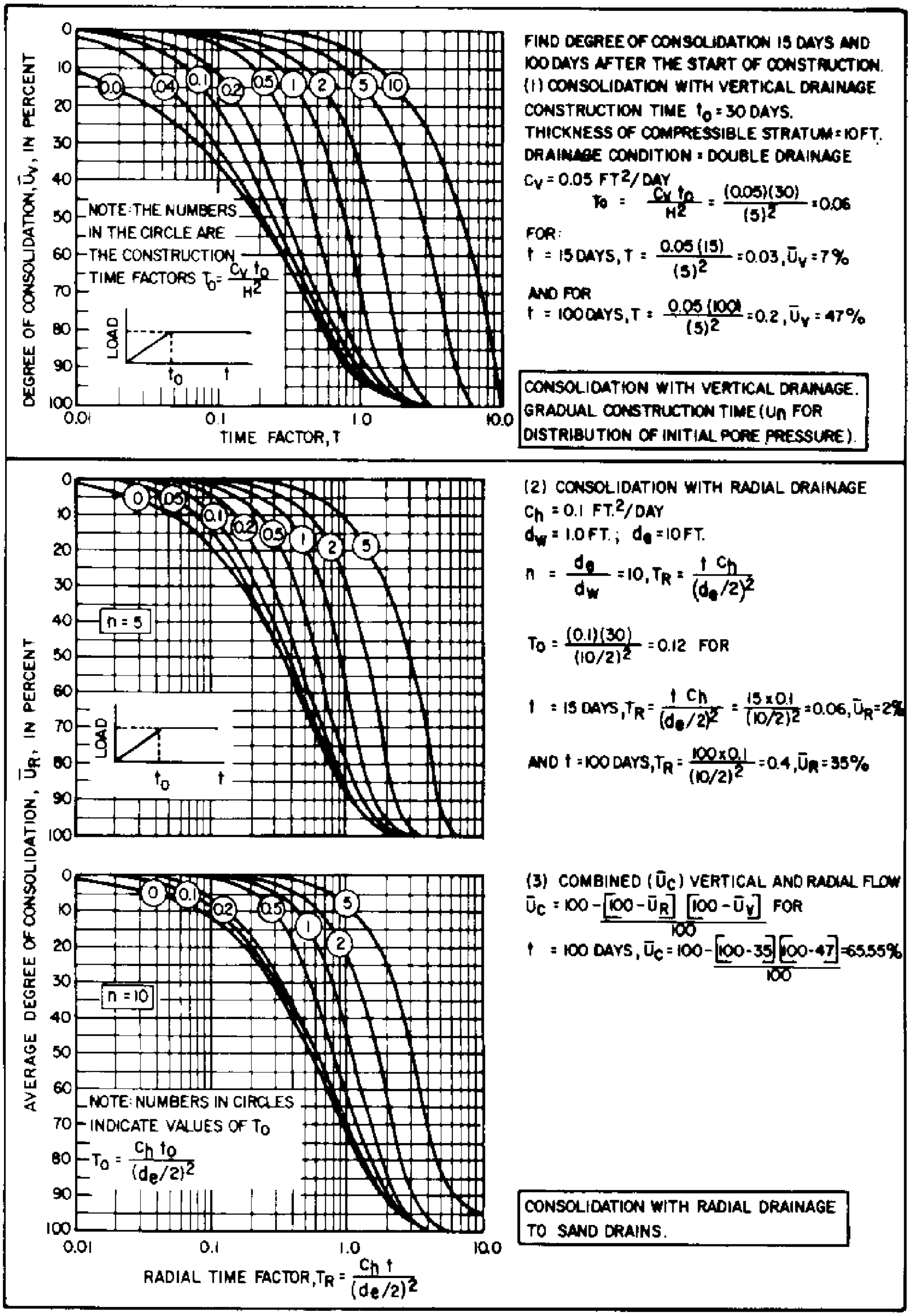
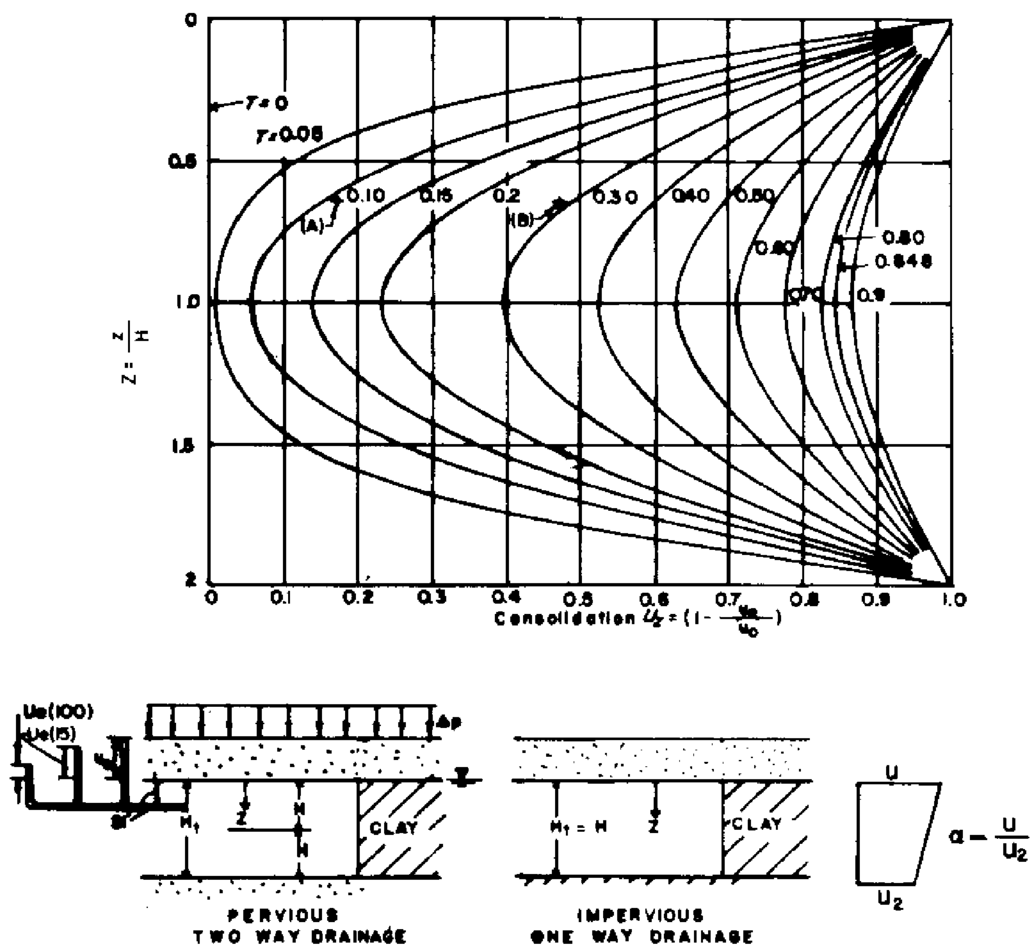


FIGURE 13
Time Rate of Consolidation for Gradual Load Application



$U_z = 1 - \frac{u_e}{u_o}$ (consolidation ratio)

u_e = Excess pore pressure at some time t

u_o = Excess pore pressure at time t = 0 (due to external loading)

FIGURE 14
Coefficient of Consolidation from Field Measurements

Example:

Thickness of clay layer H_t = 66 ft, Drainage - top & bottom

H = 66/2 = 33 ft

Depth of piezometer below top of compressible layer = 21 ft

Applied external load Δp = 1.5 KSF

Initial excess pore water pressure = u_o = Δp = 1.5 KSF

Excess pore pressure after time t_1 = 15 days, $u_e(15)$ = 20 ft = U_{et1}

Excess pore pressure after time t_2 = 100 days, $u_e(100)$ = 14 ft = U_{et2}

Piezometer measure U_o = 24 feet of water +21 ft (initial static head) for a total of 45 ft.

$$\frac{Z}{H} = \frac{21}{33} = 0.64$$

Consolidation ratio at time t_1 = 15 days = $(u_z)_{t1}$ = 1 - 20/24 = 0.17

Consolidation ratio at time t_2 = 100 days = $(u_2)_{t2}$ = 1 - 14/24 = 0.47

From above graph T_{t1} = 0.11 (point A), T_{t2} = 0.29 (point B)

$$C_v = \frac{0.29 - 0.11}{100 - 15} \times (33)^2 = 231 \text{ ft}^2\text{/day}$$

FIGURE 14 (continued)
Coefficient of Consolidation from Field Measurements

For a soil system containing n layers with properties C_{vi} (coefficient of consolidation) and H_i (layer thickness), convert the system to one equivalent layer with equivalent properties, using the following procedure:

1. Select any layer i, with properties $c_v = c_{vi}$, $H = H_i$.

2. Transform the thickness of every other layer to an equivalent thickness of a layer possessing the soil properties of layer i, as follows:

$$H'_1 = H_1 \left(\frac{c_{vi}}{c_{v1}} \right)^{1/2}$$

$$H'_2 = H_2 \left(\frac{c_{vi}}{c_{v2}} \right)^{1/2}$$

$$H'_n = H_n \left(\frac{c_{vi}}{c_{vn}} \right)^{1/2}$$

3. Calculate the total thickness of the equivalent layer:

$$H'_T = H'_2 + H'_2 + \ldots + H'_i + \ldots + H'_n$$

4. Treat the system as a single layer of thickness H'_T, possessing a coefficient of consolidation $c_v = c_{vi}$.

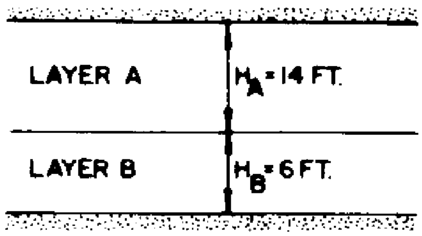
5. Determine values of percent consolidation (U) at various times (t) for total thickness (H'_T) using nomograph in Figure 11.

FIGURE 15
Procedure for Determining the Pate of Consolidation
for All Soil Systems Containing "N" layers

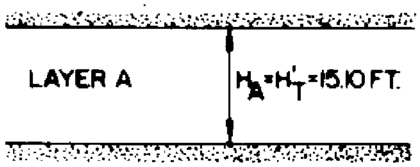
EXAMPLE OF COMPUTATION OF RATE OF CONSOLIDATION FOR A MULTI-LAYERED SYSTEM:

LAYERED SYSTEM:

ACTUAL STRATIFICATION



EQUIVALENT STRATIFICATION



KNOWN APPLICABLE SOIL PROPERTIES:

LAYER A:

$c_{vA} = 0.04$ FT.^{2}/DAY

LAYER B:

$c_{vB} = 1.20$ FT.^{2}/DAY

ASSUME: DOUBLE DRAINAGE

DETERMINATION OF EQUIVALENT LAYER THICKNESS:

1. ASSUME AN EQUIVALENT LAYER POSSESSING THE PROPERTIES OF SOIL A.
2. EQUIVALENT THICKNESS

$$H'_T = H_A + H'_B$$
$$= H_A + H_B \left(\frac{c_{vA}}{c_{vB}}\right)^{1/2}$$
$$= 14 + 6 \left(\frac{0.04}{1.20}\right)^{1/2}$$
$$= 14 + 1.10$$
$$H'_T = 15.10 \text{ FT.}$$

3. DETERMINE $\bar{U}$ FROM FIGURE 11, e.g. AT $t = 0.25$ YEARS, USING $H = (15.10)/2 = 7.55$ FT. (DRAINAGE PATH ASSUMING DOUBLE DRAINAGE) AND $c_{vA} = 0.04$ FT.^{2}/DAY, $\bar{U} = 29.1\%$

FIGURE 15 (continued)
Procedure for Determining the Rate of Consolidation
for All Soil Systems Containing "N" Layers

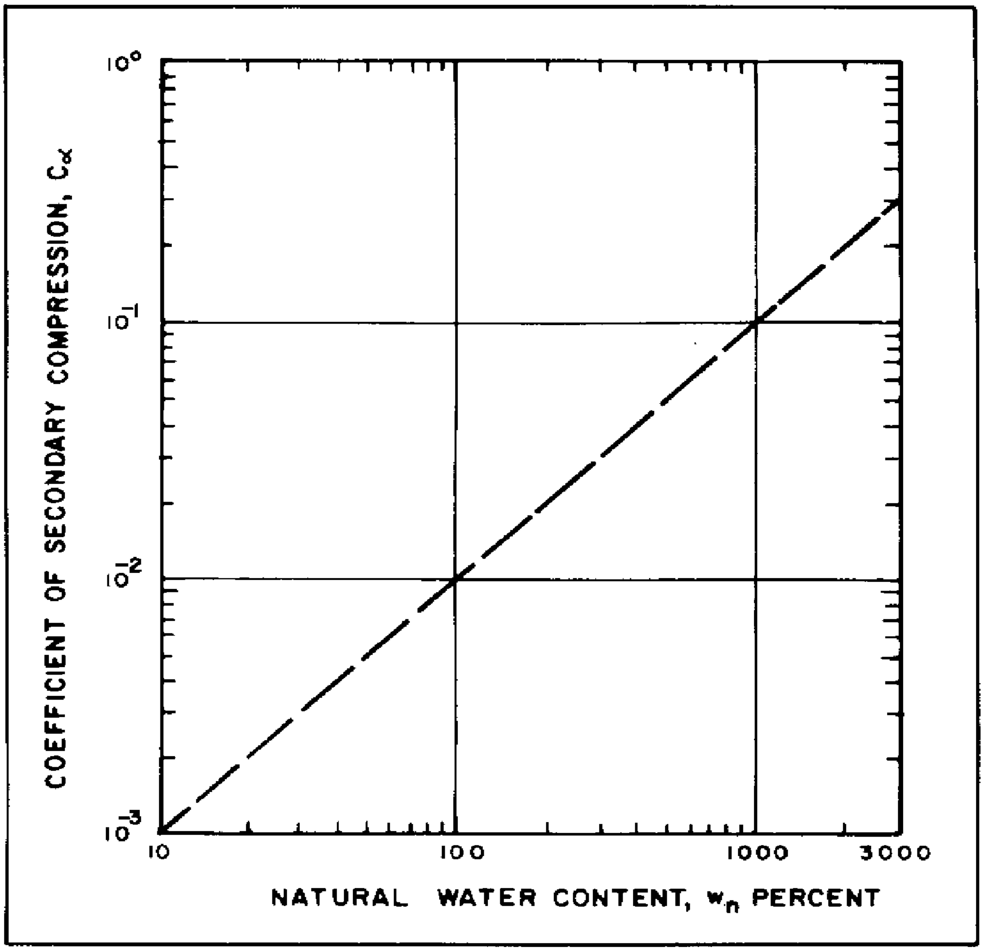


FIGURE 16
Coefficient of Secondary Compression as Related to
Natural Water Content

4. SANITARY LANDFILL. Foundations on sanitary landfills will undergo extensive settlements, both total and distortional, which are extremely difficult to predict. Settlements result not only from compression of the underlying materials, but also from the decomposition of organic matter. Gases in landfill areas are health and fire hazards. A thorough study isnecessary when utilizing sanitary landfill areas for foundations. Further guidance is given in DM-7.3, Chapter 3.

5. PEAT AND ORGANIC SOILS. Settlements in these soils are computed in a similar manner as for fine-grained soils. However, the primary consolidation takes place rapidly and the secondary compression continues for a long period of time and contributes much more to the total settlement.

Section 5. TOLERABLE AND DIFFERENTIAL SETTLEMENT

1. APPLICATIONS. For an important structure, compute total settlement at a sufficient number of points to establish the overall settlement pattern. From this pattern, determine the maximum scope of the settlement profile or the greatest difference in settlement between adjacent foundation units.

2. APPROXIMATE VALUES. Because of natural variation of soil properties and uncertainty on the rigidity of structure and thus actual loads transmitted to foundation units, empirical relationships have been suggested to estimate the differential settlements (or angular distortion) in terms of total settlement (see Reference 11, Structure Soil Interaction, by Institution of Civil Engineers). Terzaghi and Peck (Reference 8, page 489) suggested that for footings on sand, differential settlement is unlikely to exceed 75% of the total settlement. For clays, differential settlement may in some cases approach the total settlement.

3. TOLERABLE SETTLEMENT

a. Criteria. Differential settlements and associated rotations and tilt may cause structural damage and could impair the serviceability and function of a given structure. Under certain conditions, differential settlements could undermine the stability of the structure and cause structural failure. Table 4 (Reference 12, Allowable Settlements of Structures, by Bjerrum) provides some guidelines to evaluate the effect of settlement on most structures. Table 5 provides guidelines for tanks and other facilities.

b. Reduction of Differential Settlement Effects. For methods of reducing or accelerating consolidation settlements, see Section 6. Settlement that can be completed during the early stages of construction, before placing sensitive finishes, generally will not contribute to structural distress. In buildings with light frames where large differential settlements may not harm the frame, make special provisions to avoid damage to utilities or operating equipment. Isolate sensitive equipment, such as motor-generator sets within the structure, on separate rigidly supported foundations. Provide flexible couplings for utility lines at critical locations.

TABLE 4
Tolerable Settlements for Building

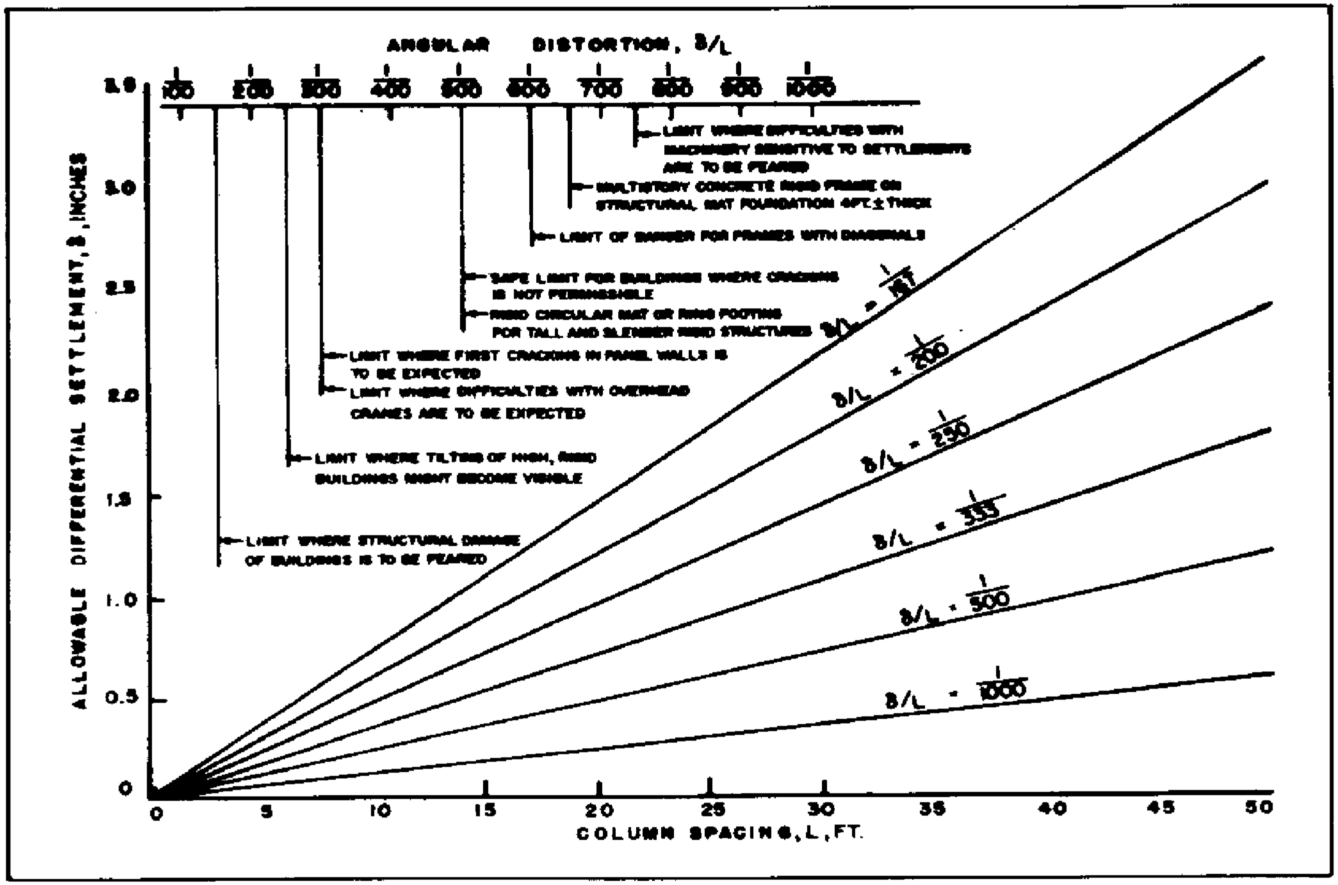
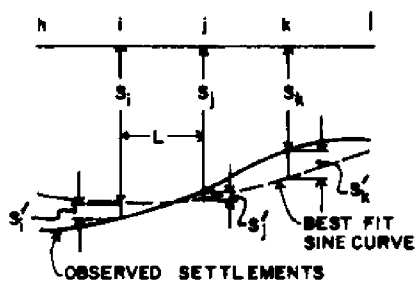


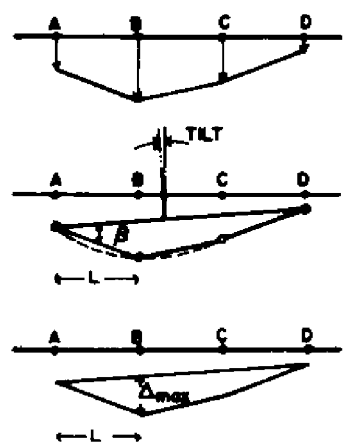
TABLE 5
Tolerable Differential Settlement for Miscellaneous Structures

STRUCTURE		TOLERABLE DISTORTION $\frac{\Delta_{max}}{L}$ or β
A. UNREINFORCED LOAD BEARING WALLS (L and H are respectively length and height of the wall from top of footing)	SAGGING FOR L/H < 3	$\frac{\Delta_{max}}{L}$ = 1/3500 to 1/2500
	FOR L/H > 5	$\frac{\Delta_{max}}{L}$ = 1/2000 to 1/1250
	HOGGING FOR L/H = 1	$\frac{\Delta_{max}}{L}$ = 1/5000
	FOR L/H = 5	$\frac{\Delta_{max}}{L}$ = 1/2500
B. JOINTED RIGID CONCRETE PRESSURE CONDUITS (MAXIMUM ANGLE CHANGE AT JOINT 2 TO 4 TIMES AVERAGE SLOPE OF SETTLEMENT PROFILE. LONGITUDINAL EXTENSION AFFECTS DAMAGE.)		1/65
C. CIRCULAR STEEL PETROLEUM OR FLUID STORAGE TANKS.		β < 1/300 β' = 1/500 to 1/300



$$\beta = \frac{S_i - S_j}{L}$$

$$\beta' = \left(S_j' - \frac{S_i' + S_k'}{2}\right)\frac{1}{L}$$



FOUNDATION MOVEMENTS

4. EFFECT OF STRUCTURE RIGIDITY. Computed differential settlement is less accurate than computed total or average settlement because the interaction between the foundation elements and the supporting soil is difficult to predict. Complete rigidity implies uniform settlement and thus no differential settlement. Complete flexibility implies uniform contact pressure between the mat and the soil. Actual conditions are always in between the two extreme conditions. However, depending on the magnitude of relative stiffness as defined below, mats can be defined as rigid or flexible for practical purposes.

a. Uniformly Loaded Circular Raft. In the case where the raft has a frictionless contact with an elastic half space (as soil is generally assumed to represent), the relative stiffness is defined as

[retrieve Equation]

R = radius of the raft, t = thickness of raft, subscripts r and s refer to raft and soil, [upsilon] = Poission's ratio and E = Young's modulus.

For K+r, < /= 0.08, raft is considered flexible and for K+r, < /= 5.0 raft is considered rigid.

For intermediate stiffness values see Reference 13, Numerical Analyses of Uniformly Loaded Circular Rafts on Elastic Layers of Finite Depth, by Brown.

b. Uniformly Loaded Rectangular Raft. For frictionless contact between the raft and soil, the stiffness factor is defined as:

[retrieve Equation]

B = width of the foundation. Other symbols are defined in (a).

For K+r, < /= 0.05, raft is considered flexible and for K+r, >/= 10, raft is considered rigid.

For intermediate stiffness values see Reference 14, Numerical Analysis of Rectangular Raft on Layered Foundations, by Frazer and Wardle.

Section 6. METHODS OF REDUCING OR ACCELERATING SETTLEMENT

1. GENERAL. See Table 6 for methods of minimizing consolidation settlements. These include removal or displacement of compressible material and preconsolidation in advance of final construction.

2. REMOVAL OF COMPRESSIBLE SOILS. Consider excavation or displacement of compressible materials for stabilization of fills that must be placed over soft strata.

TABLE 6

Methods of Reducing or Accelerating Settlement or Coping with Settlement

Method	Comment
Procedures for linear fills on swamps or compressible surface stratum:	
Excavation of soft material....	When compressible foundation soils extend to depth of about 10 to 15 ft, it may be practicable to remove entirely. Partial removal is combined with various methods of displacing remaining soft material.
Displacement by weight of fill.	Complete displacement is obtained only when compressible foundation is thin and very soft. Weight displacement is combined with excavation of shallow material.
Jetting to facilitate displacement................	For a sand or gravel fill, jetting within the fill reduces its rigidity and promotes shear failure to displace soft foundation. Jetting within soft foundation weakens it to assist in displacement.
Blasting by trench or shooting methods.....................	Charge is placed directly in front of advancing fill to blast out a trench into which the fill is forced by the weight of surcharge built up at its point. Limited to depths not exceeding about 20 ft.
Blasting by relief method......	Used for building up fill on an old roadway or for fills of plastic soil. Trenches are blasted at both toes of the fill slopes, relieving confining pressure and allowing fill to settle and displace underlying soft materials.
Blasting by underfill method...	Charge is placed in soft soil underlying fill by jetting through the fill at a preliminary stage of its buildup. Blasting loosens compressible material, accelerating settlement and facilitating displacement to the sides. In some cases relief ditches are cut or blasted at toe of the fill slopes. Procedure is used in swamp deposits up to 30 ft thick.

TABLE 6 (continued)

Methods of Reducing or Accelerating Settlement or Coping with Settlement

Method	Comment
Procedures for preconsolidation of soft foundations:	
Surcharge fill..............	Used where compressible stratum is relatively thin and sufficient time is available for consolidation under surcharge load. Surcharge material may be placed as a stockpile for use later in permanent construction. Soft foundation must be stable against shear failure under surcharge load.
Accelerating consolidation by vertical drains...............	Used where tolerable settlement of the completed structure is small, where time available for preconsolidation is limited, and surcharge fill is reasonably economical. Soft foundation must be stable against shear failure under surcharge load.
Vertical sand drains with or without surcharge fill........	Used to accelerate the time for consolidation by providing shorter drainage paths.
Wellpoints placed in vertical sand drains..................	Used to accelerate consolidation by reducing the water head, thereby permitting increased flow into the sand drains. Particularly useful where potential instability of soft foundation restricts placing of surcharge or where surcharge is not economical.
Vacuum method................	Variation of wellpoint in vertical sand drain but with a positive seal at the top of the sand drain surrounding the wellpoint pipe. Atmospheric pressure replaces surcharge in consolidating soft foundations.
Balancing load of structure by excavation................	Utilized in connection with mat or raft foundations on compressible material or where separate spread footings are founded in suitable bearing material overlying compressible stratum. Use of this method may eliminate deep foundations, but it requires very thorough analysis of soil compressibility and heave.

a. Removal by Excavation. Organic swamp deposits with low shear strength and high compressibility should be removed by excavation and replaced by controlled fill. Frequently these organic soils are underlain by very loose fine sands or silt or soft clayey silts which may be adequate for the embankment foundation and not require replacement.

Topsoil is usually stripped prior to placement of fills; however, stripping may not be required for embankments higher than 6 feet as the settlement from the upper 1/2 foot of topsoil is generally small and takes place rapidly during construction period. However, if the topsoil is left in place, the overall stability of the embankment should be checked assuming a failure plane through the topsoil using the methods of Chapter 7.

b. Displacement. Partial excavation may be accompanied by displacement of the soft foundation by the weight of fill. The advancing fill should have a steep front face. The displacement method is usually used for peat and muck deposits. This method has been used successfully in a few cases for soft soils up to 65 feet deep. Jetting in the fill and various blasting methods are used to facilitate displacement. Fibrous organic materials tend to resist displacement resulting in trapped pockets which may cause differential settlement.

3. BALANCING LOAD BY EXCAVATION. To decrease final settlement, the foundation of heavy structures may be placed above compressible strata within an excavation that is carried to a depth at which the weight of overburden, removed partially or completely, balances the applied load.

a. Computation of Total Settlement. In this case, settlement is derived largely from recompression. The amount of recompression is influenced by magnitude of heave and magnitude of swell in the unloading stage.

b. Effect of Dewatering. If drawdown for dewatering extends well below the planned subgrade, heave and consequent recompression are decreased by the application of capillary stresses. If groundwater level is restored after construction, the load removed equals the depth of excavation times total unit weight of the soil. If groundwater pressures are to be permanently relieved, the load removed equals the total weight of soil above the original water table plus the submerged weight of soil below the original water table. Calculate effective stresses as described in Figure 2, and consolidation under structural loads as shown in Figure 3.

4. PRECONSOLIDATION BY SURCHARGE. This procedure causes a portion of the total settlement to occur before construction. It is used primarily for fill beneath paved areas or structures with comparatively light column loads. For heavier structures, a compacted fill of high rigidity may be required to reduce stresses in compressible foundation soil (see DM-7.2, Chapter 2).

a. Elimination of Primary Consolidation. Use Figure 17 to determine surcharge load and percent consolidation under surcharge necessary to eliminate primary consolidation under final load. This computation assumes that the rate of consolidation under the surcharge is equal to that under final load.

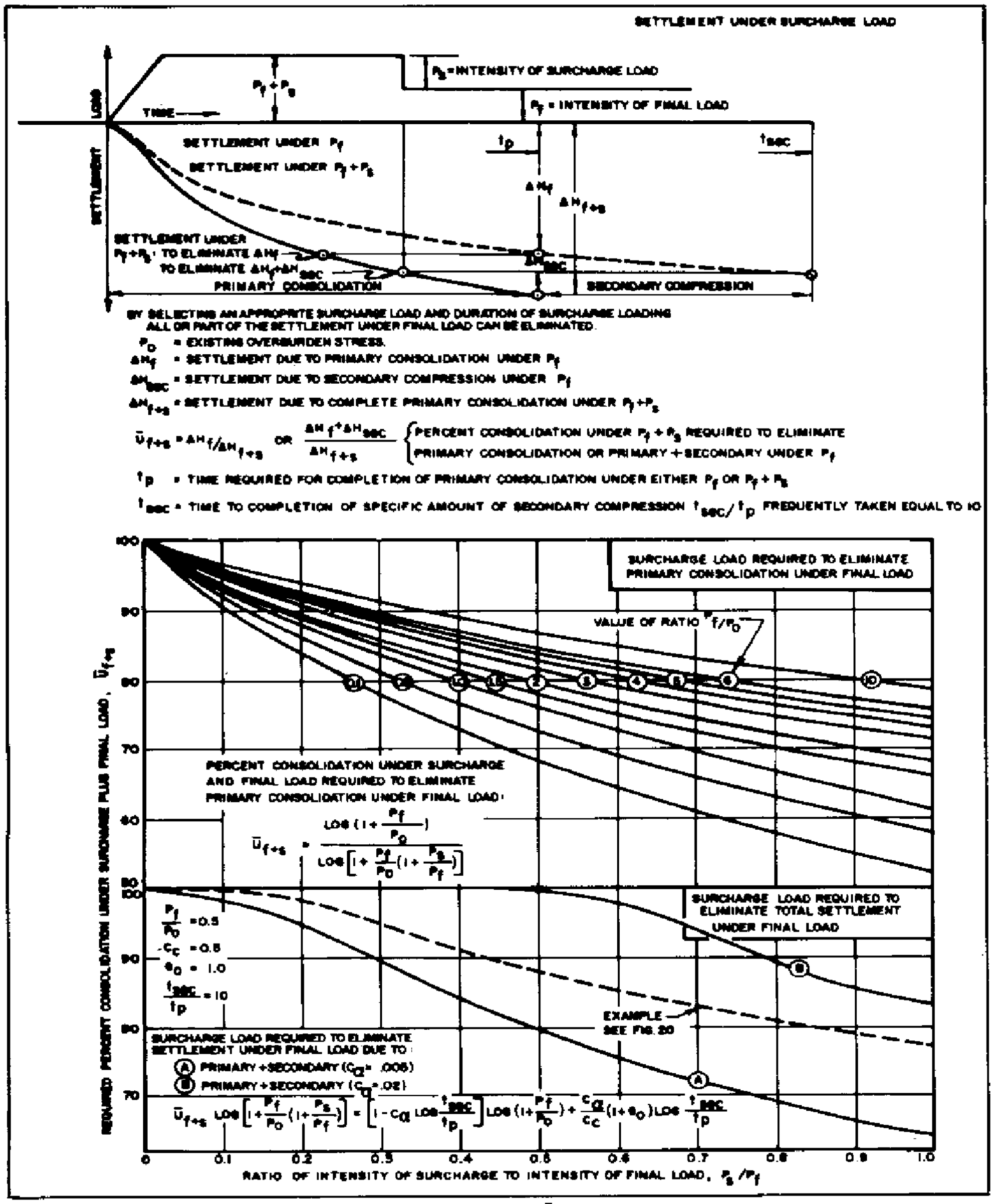


FIGURE 17
Surcharge Load Required to Eliminate Settlement Under Final Load

b. Elimination of Secondary Consolidation. Use the formula in the bottom panel of Figure 17 to determine surcharge load and percent consolidation under surcharge required to eliminate primary consolidation plus a specific secondary compression under final load.

c. Limitations on Surcharge. In addition to considerations of time available and cost, the surcharge load may induce shear failure of the soft foundation soil. Analyze stability under surcharge by methods of Chapter 7.

5. VERTICAL DRAINS. These consist of a column of pervious material placed in cylindrical vertical holes in the compressible stratum at sufficiently close spaces so that the horizontal drainage path is less than the vertical drainage path. All drains should be connected at the ground surface to a drainage blanket. Vertical drains are utilized in connection with fills supporting pavements or low- to moderate-load structures and storage tanks. Common types of vertical drains are shown in Table 7 (Reference 15, Use of Precompression and Vertical Sand Drains for Stabilization of Foundation Soils, by Ladd). Sand drains driven with a closed-end pipe produce the largest displacement and disturbance in the surrounding soil and thus their effectiveness is reduced.

a. Characteristics. Vertical drains accelerate consolidation by facilitating drainage of pore water but do not change total compression of the stratum subjected to a specific load. Vertical drains are laid out in rows, staggered, or aligned to form patterns of equilateral triangles or squares. See Figure 18 for cross-section and design data for typical installation for sand drains.

b. Consolidation Rate. Time rate of consolidation by radial drainage of pore water to vertical drains is defined by time factor curves in upper panel of Figure 10. For convenience, use the nomograph of Figure 19 to determine consolidation time rate. Determine the combined effect of vertical and radial drainage on consolidation time rate as shown in the example in Figure 10.

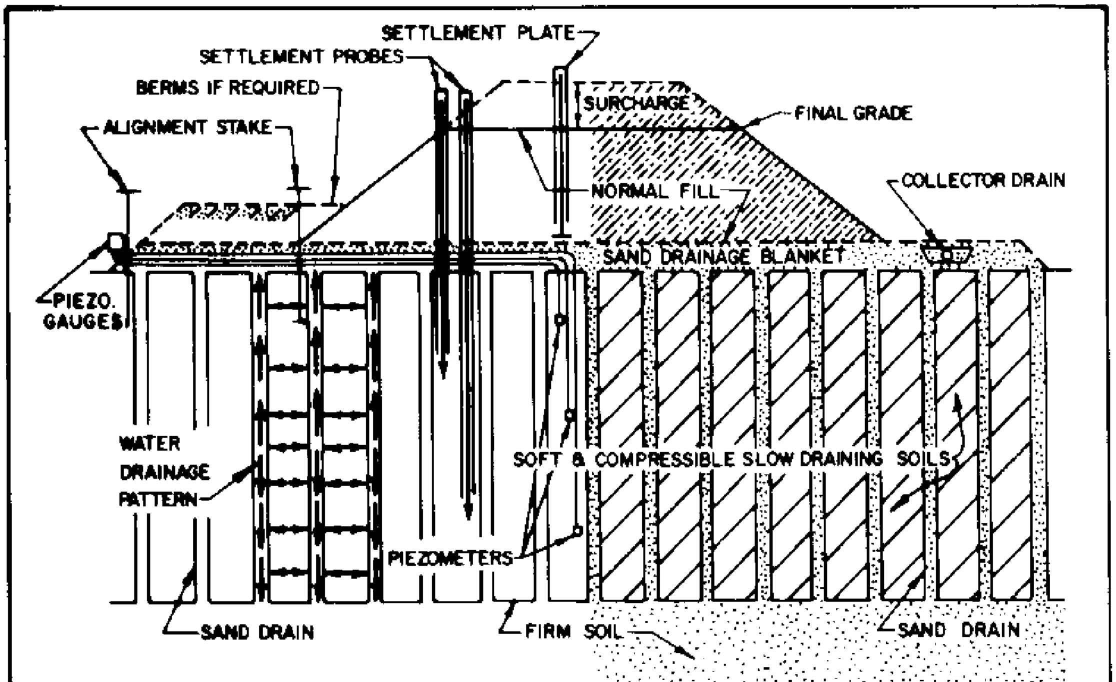
c. Vertical Drain Design. See Figure 20 for an example of design. For a trial selection of drain diameter and spacing, combine percent consolidation at a specific time from vertical drainage with percent consolidation for radial drainage to the drain. This combined percent consolidation U+c, is plotted versus elapsed time for different drain spacing in the center panel of Figure 20. Selection of drain spacing depends on the percent consolidation required prior to start of structure, the time available for consolidation, and economic considerations.

d. Allowance for Smear and Disturbance. In cases where sand drain holes are driven with a closed-end pipe, soil in a surrounding annular space one-third to one-half the drain diameter in width is remolded and its stratification is distorted by smear. Smear tends to reduce the horizontal permeability coefficient, and a correction should be made in accordance with Figure 21.

TABLE 7
Common Types of Vertical Drains

General Type	Sub-type	Typical Installation	
		d_w	s
1. Driven Sand Drain	Closed end mandrel	18± in	5 - 20 ft
2. Augered Sand Drain	(a) Screw type auger	6 - 30 in	-
	(b) Continuous flight hollow stem auger	18 in	5 - 20 ft
3. Jetted Sand Drain	(a) Internal jetting	18 in	5 - 20 ft
	(b) Rotary jet	12 - 18 in	5 - 20 ft
	(c) Dutch jet-bailer	12 in	4 - 16 ft
4. "Paper" Drain	(a) Kjellman cardboard wick	0.1± in by 4± in	1.5^+ - 4^+ ft
	(b) Cardboard coated plastic wick	slightly thicker	-
5. Fabric Encased Sand Drain	(a) Sandwick	2.5 - 3 in	4 - 12 ft
	(b) Fabridrain	5 in	-

d_w = diameter of drain, s = drain spacing



TYPICAL SAND DRAIN INSTALLATION

DIAMETER OF DRAINS RANGES FROM 6 TO 30 IN., GENERALLY BETWEEN 16 AND 20 IN.

SPACING OF DRAINS RANGES FROM 6 TO 20 FT ON CENTER, GENERALLY BETWEEN 6 AND 10 FT.

PRINCIPAL METHODS FOR INSTALLING DRAINS ARE CLOSED OR OPEN MANDRELS ADVANCED BY DRIVING OR JETTING, OR ROTARY DRILLING WITH OR WITHOUT JETS. DRIVING CLOSED MANDREL IS THE MOST COMMON.

DRAIN BACKFILL MATERIAL SHOULD HAVE SUFFICIENT PERMEABILITY TO DISCHARGE PORE WATER FLOW ANTICIPATED, BUT USUALLY DOES NOT MEET FILTER REQUIREMENTS AGAINST FOUNDATIONS SOILS. CLEAN SANDS WITH NO MORE THAN 3% BY WEIGHT PASSING No. 200 SIEVE IS USUALLY SUITABLE. A TYPICAL GRADATION IS AS FOLLOWS:

SIEVE No.	4	16	50	100	200
% FINER BY WEIGHT:	90-100	40-85	2-30	0-7	0-3

SAND DRAINAGE BLANKET MATERIAL IS SIMILAR TO THAT USED FOR DRAIN BACKFILL. IN SOME CASES GRAVEL WINDROWS OR PERFORATED, CORRUGATED, METAL PIPE ARE PLACED IN DRAINAGE BLANKET TO REDUCE HEAD LOSS IN DRAINAGE BLANKET. LONGITUDINAL DITCH OR COLLECTOR DRAIN MAY BE PLACED AT TOE. GRANULAR WORKING MAT IS SOMETIMES PLACED BELOW DRAINAGE BLANKET TO SUPPORT EQUIPMENT.

SURCHARGE LOAD IS PLACED TO REDUCE OR ELIMINATE POSTCONSTRUCTION CONSOLIDATION BENEATH NORMAL FILL. GENERALLY THE SURCHARGE LOAD IS NO MORE THAN ABOUT 30% OF NORMAL EMBANKMENT LOAD.

FIELD CONTROL DEVICES:

PIEZOMETERS OF STANDPIPE OR CLOSED SYSTEM TYPE TO OBSERVE PORE WATER PRESSURES;

SETTLEMENT PLATES, MINIMUM 3 FT SQUARE, PLACED AT BASE OF FILL TO RECORD TOTAL SETTLEMENT;

SETTLEMENT PROBES DRIVEN OR AUGERED INTO FOUNDATION STRATUM TO MEASURE COMPRESSION WITHIN FOUNDATION.

ALIGNMENT STAKES, T-SHAPED STAKES PLACED AT OR OUTSIDE EMBANKMENT TOE TO OBSERVE LATERAL MOVEMENT AND HEAVE.

FIGURE 18
Data for Typical Sand Drain Installation

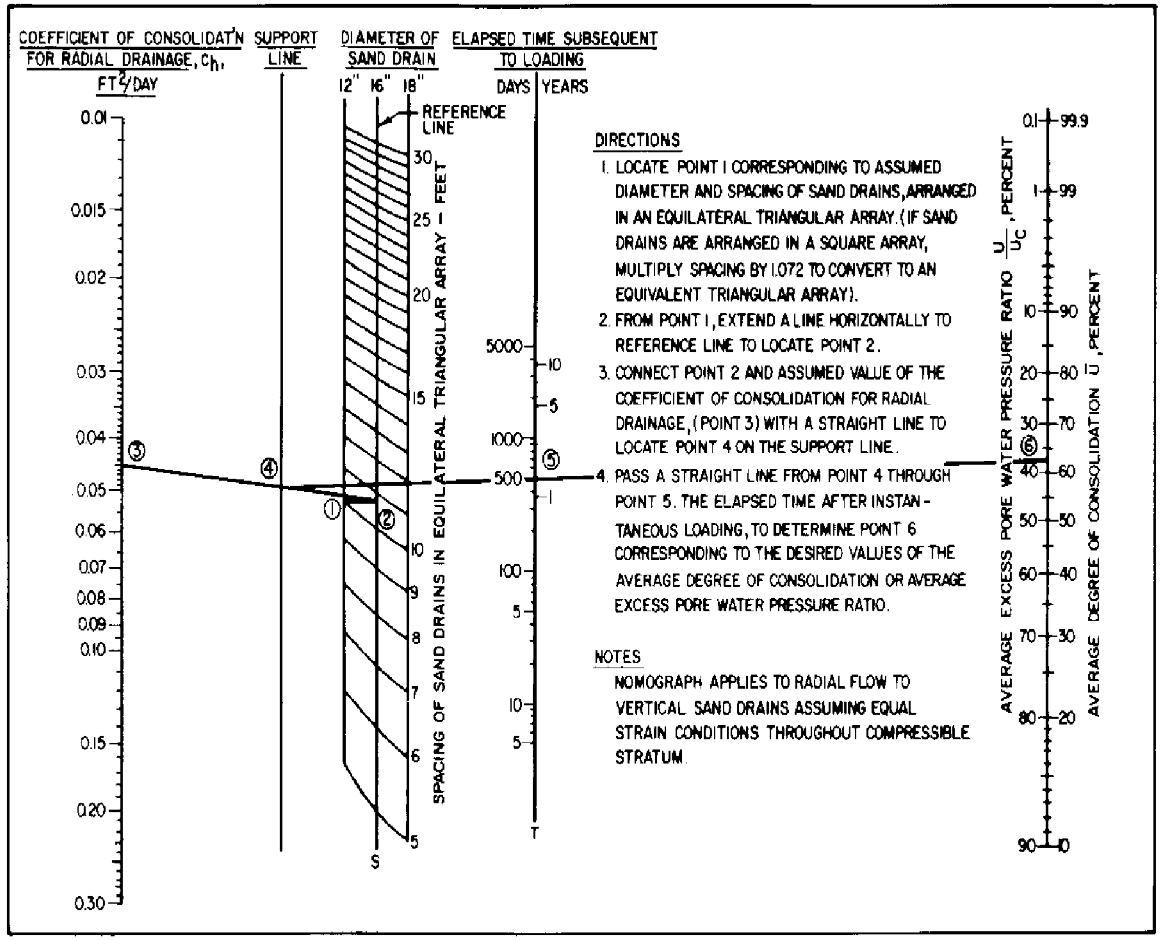


FIGURE 19
Nomograph for Consolidation with Radial Drainage to Vertical Sand Drain

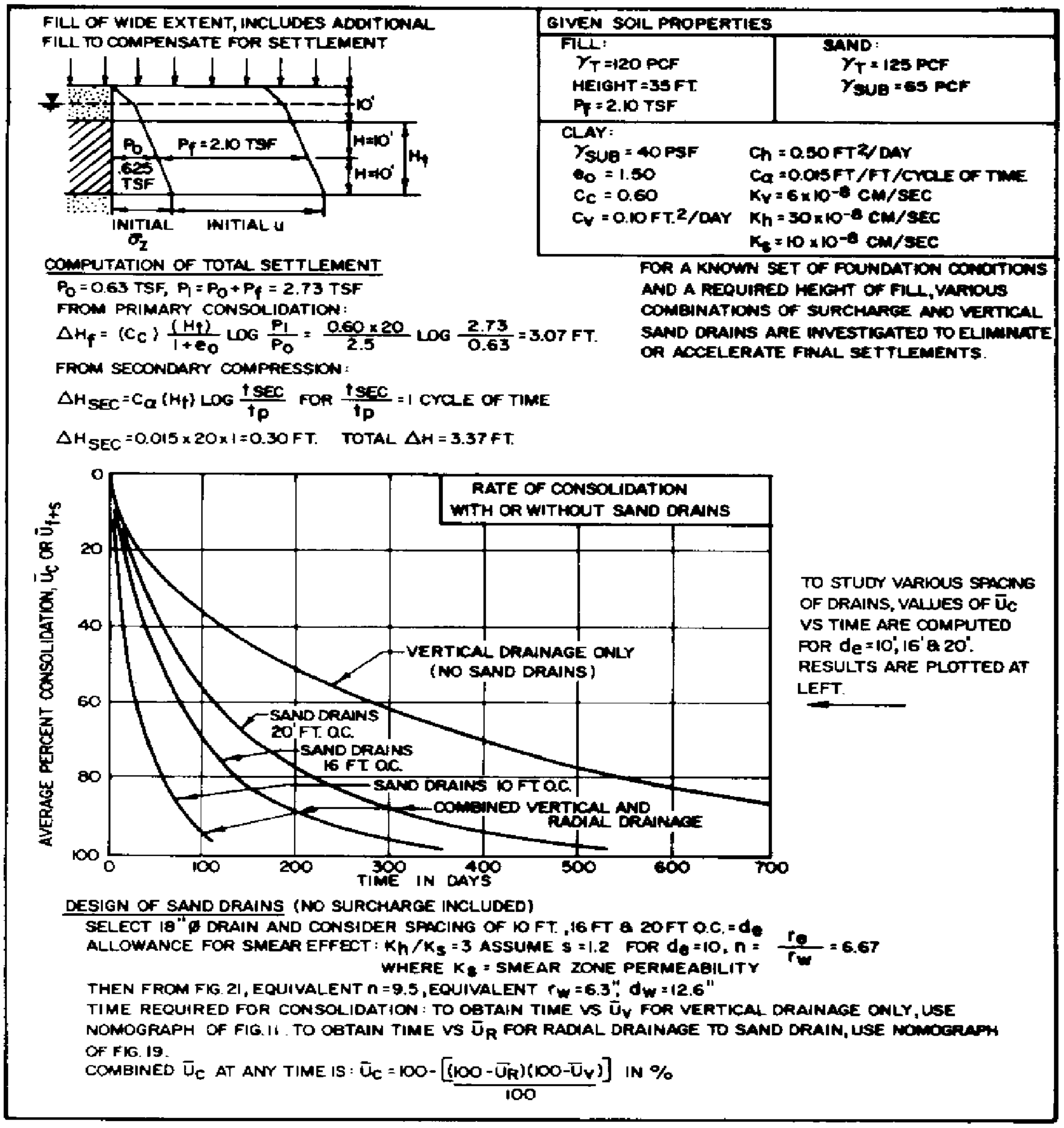
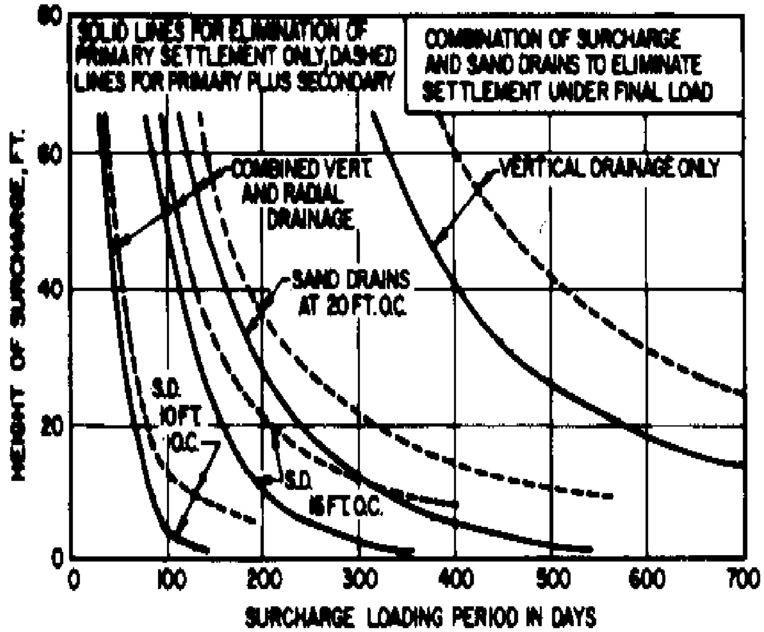


FIGURE 20
Example of Surcharge and Sand Drain Design



SELECTION OF SURCHARGE HEIGHT

$\Delta H_f = 3.07'$, $\Delta H_{SEC.} = 0.30'$, $P_f/P_0 = 3.38$

TO ELIMINATE SETTLEMENT UNDER P_f, $\bar{U}_C$ IS TAKEN EQUAL TO $\bar{U}_{f+s}$, $\bar{U}_{f+s} = \dfrac{\Delta H_f}{\Delta H_{f+s}}$ OR $\dfrac{\Delta H_f + \Delta H_{SEC}}{\Delta H_{f+s}}$

RELATION OF $\bar{U}_{f+s}$ AND TIME IS GIVEN ABOVE FOR VARIOUS DRAIN SPACINGS.

SURCHARGE P_s FOR VALUES OF $\bar{U}_{f+s} = \dfrac{\Delta H_f}{\Delta H_{f+s}}$ IS GIVEN IN FIG. 17

SURCHARGE P_s FOR VALUES OF $\bar{U}_{f+s} = \dfrac{\Delta H_f + \Delta H_{SEC}}{\Delta H_{f+s}}$ IS GIVEN BY FORMULA IN FIG. 17

USING THESE RELATIONSHIPS, P_s (EXPRESSED AS HEIGHT OF SURCHARGE) REPLACES $\bar{U}_{f+s}$ IN FIGURE 17.

COMBINATION OF SAND DRAIN AND SURCHARGE IS SELECTED BASED ON TIME AVAILABLE AND COMPARATIVE COSTS.

FIGURE 20 (continued)
Example of Surcharge and Sand Drain Design

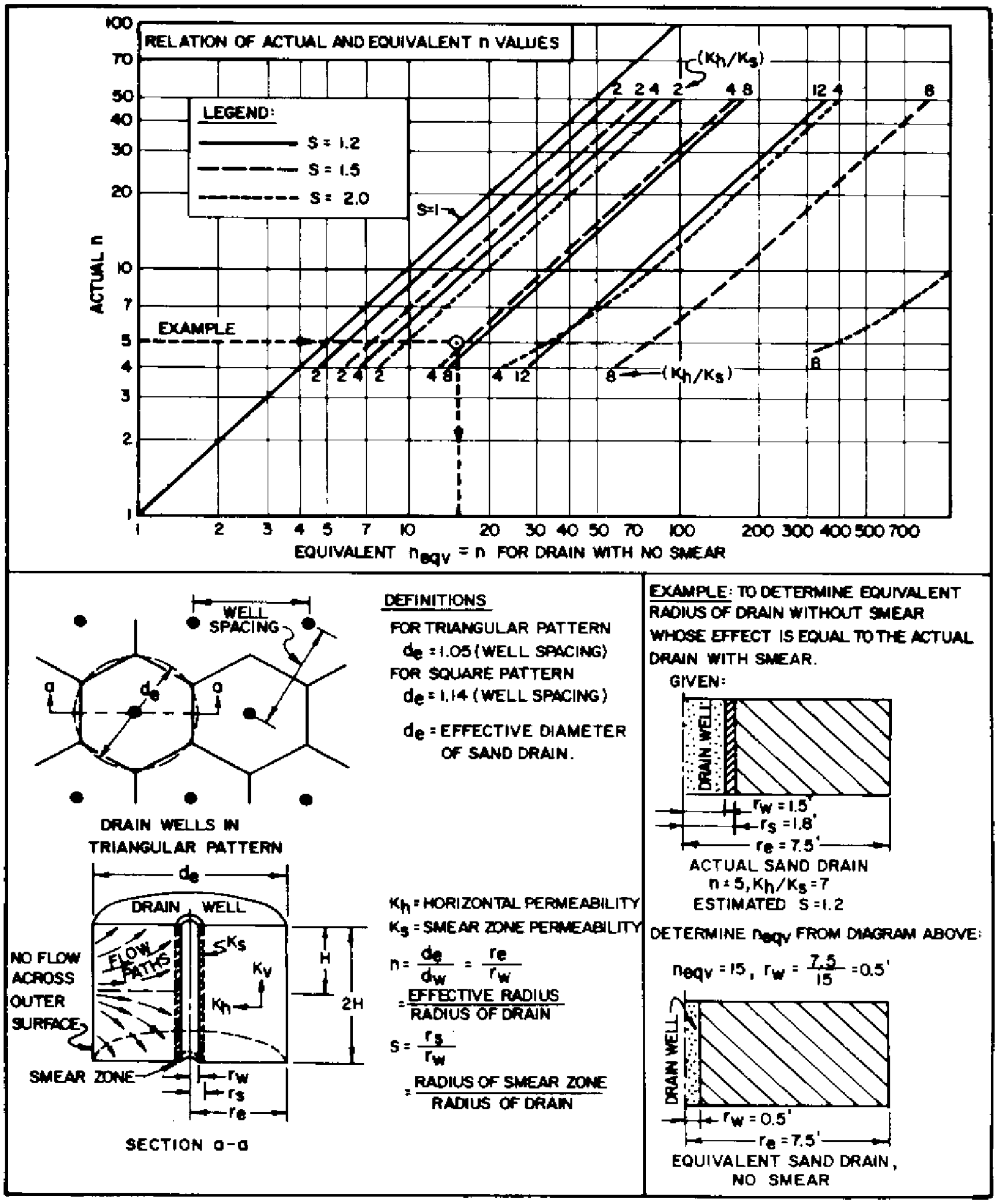


FIGURE 21
Allowance for Smear Effect in Sand Drain Design

e. Sand Drains Plus Surcharge. A surcharge load is normally placed above the final fill level to accelerate the required settlement. Surcharge is especially necessary when the compressible foundation contains material in which secondary compression predominates over primary consolidation. The percent consolidation under the surcharge fill necessary to eliminate a specific amount of settlement under final load is determined as shown in the lowest panel of Figure 20.

f. General Design Requirements. Analyze stability against foundation failure by the methods of Chapter 7, including the effect of pore pressures on the failure plane. Determine allowable buildup of pore pressure in the compressible stratum as height of fill is increased.

(1) Horizontal Drainage. For major installation investigate in detail the horizontal coefficient of consolidation by laboratory tests with drainage in the horizontal direction, or field permeability tests to determine horizontal permeability.

(2) Consolidation Tests. Evaluate the importance of smear or disturbance by consolidation tests on remolded samples. For sensitive soils and highly stratified soils, consider nondisplacement methods for forming drain holes.

(3) Drainage Material. Determine drainage material and arrangement to handle maximum flow of water squeezed from the compressible stratum in accordance with Chapter 6.

g. Construction Control Requirements. Control the rate of fill rise by installing piezometer and observing pore pressure increase for comparison with pore pressure values compatible with stability. Check anticipated rate of consolidation by pore pressure dissipation and settlement measurements.

Section 7. ANALYSIS OF VOLUME EXPANSION

1. CAUSES OF VOLUME EXPANSION. Volume expansion is caused by (a) reduction of effective stresses, (b) mineral changes, and (c) formation and growth of ice lenses. Swell with decrease of effective stress is a reverse of the consolidation process. For description of swelling problems and suggested treatment, see Table 8. Where highly preconsolidated plastic clays are present at the ground surface, seasonal cycles of rainfall and desiccation produce volume changes. The most severe swelling occurs with montmorillinite clays although, in an appropriate climate, any surface clay of medium to high plasticity with relatively low moisture content can heave. For estimation of swell potential see Chapter 1, Section 6.

2. MAGNITUDE OF VOLUME EXPANSION. Figure 22 outlines a procedure for estimating the magnitude of swelling that may occur when footings are built on expansive clay soils. This figure also indicates a method of determining the necessary undercut to reduce the heave to an acceptable value. Further guidance for foundations on expansive soils is contained in DM-7.3, Chapter 3.

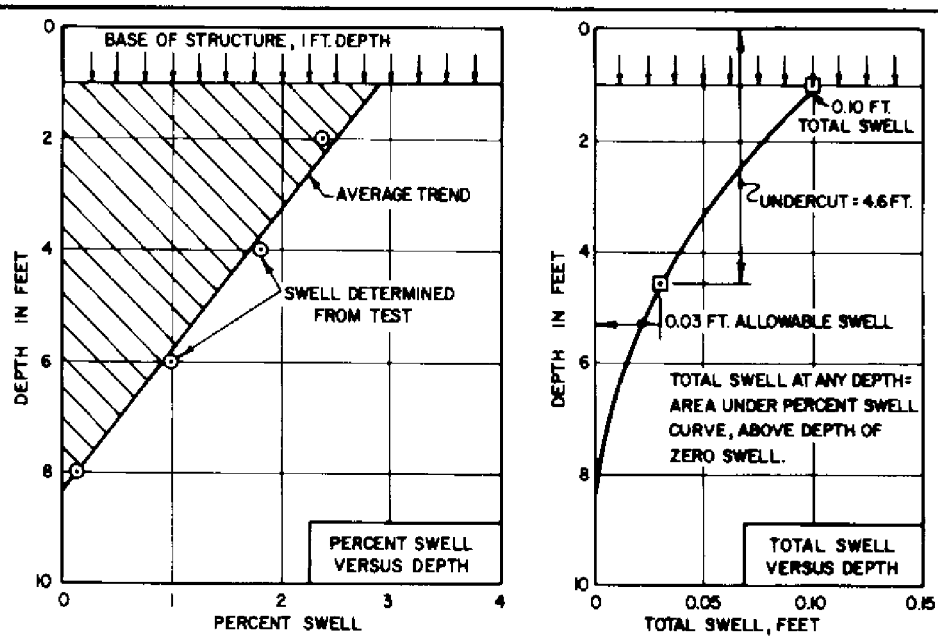
TABLE 8

Heave From Volume Change

Conditions and materials	Mechanism of heave	Treatment
Reduction of effective stress of overburden:		
Temporary reduction of effective stress by excavation for structure foundation in preconsolidated clays.	Soil swells in accordance with laboratory e-p curves. Heave is maximum at center of excavation. Total potential heave may not have occurred by time the load is reapplied. Final structural load will recompress foundation materials.	Provide drainage for rapid collection of surface water. Avoid disturbance to subgrade by placing 4-in.-thick working mat of lean concrete immediately after exposing subgrade. Heave is minimized if the groundwater is drawn down 3 or 4 ft below base of excavation at its center to maintain capillary stresses.
Permanent reduction of effective stress by excavation in chemically inert, uncemented clay-shale or shale.	In sound shale where water cannot obtain access to the shale, swelling may be insignificant.	Protect shale from wetting and drying during excavation by limiting area opened at subgrade and with concrete working mat. Pour concrete floors and foundations directly on protected shale with no underfloor drainage course. Backfill around walls with impervious soils to prevent access of water. Provide proper surface drainage and paving if necessary to avoid infiltration.
	For hydraulic structures or construction below the ground water table, reduction of effective stresses will cause permanent heave in accordance with laboratory e-p curves. Alternate wetting and drying during excavation increases swelling potential.	Where an increase in water content is probable, special structural designs must be considered. These include (1) anchoring or rock bolting the floor to a depth in shale that provides suitable hold down against swelling pressures; (2) a floor supported on heavily loaded column footings with an opening or compressible filler beneath floors; and (3) a mat foundation designed to resist potential swelling pressures. In any case, excavation in the shale should be protected by sealing coats or working mat immediately after exposure at subgrade.
Reduction of effective stress of overburden and release of capillary stress:		
Construction of earth dams of heavily compacted plastic clays.	Intrusion of seepage from reservoir releases capillary pressures and reduces effective stress of overburden and may produce swelling leading to sloughing of the slopes. Most critical material are CH clays with swelling index exceeding 0.07. Compaction at relatively low water contents, where the water deficiency in the clay mineral lattice is high and the degree of saturation is low, will accentuate swelling.	Avoid placing highly plastic fill on or near embankment slopes. Compact clays at a relatively high moisture content consistent with strength and compressibility requirements. Avoid overcompaction to an unnecessarily high dry unit weight.
Construction of structural fill for light buildings of compacted plastic clay.	Rise of groundwater, seepage, leakage, or elimination of surface evaporation increases degree of saturation and reduces effective stress, leading to expansion.	Compact clays as wet as practicable consistent with compressibility requirements. Avoid overcompaction of general fill and undercompaction of backfill at column footings or in utility trenches which would accentuate differential movements. Stabilization of compacted fills with various salt admixtures reduces swelling potential by increasing ion concentration in pore water.

TABLE 8 (continued)
Heave From Volume Change

Conditions and materials	Mechanism of heave	Treatment
Changes of capillary stresses:		
Construction of light buildings on surface strata of highly preconsolidated clays in temperate climates subject to substantial seasonal fluctuations in rainfall. (Southern England, as an example.)	Seasonal movements 1 or 2 in. upwards and downwards occur within the upper 3 to 5 ft. Settlement occurs in early summer and expansion in the fall. Caused by change of capillary stresses produced by transpiration to nearby trees, plant, or grass cover surrounding the structure. Movements are maximum at edge of building. Groundwater is shallow. Change of capillary stresses by evaporation is not of prime importance.	Light reinforcing or stiffening minimize effects in small houses. Basements carried to usual depths usually eliminate movements.
Construction of light buildings on clays of high activity, highly preconsolidated with fractures and slickensides, in climate where hot summers alternate with wet winters. (South-central Texas for example.)	Even in the absence of vegetal cover, seasonal cycles of settlement and heave occur because of the alternate increase and release of capillary stresses. Buildings constructed during wet season may undergo small but nonuniform settlement beneath exterior footings. Buildings constructed in the dry season undergo uneven heave up to 3 or 4 in. maximum, distributed irregularly over the structure.	Support light footings and slabs on compacted, coarse-grained fill about 4 to 6 ft thick. Pave peripheral areas to minimize subsoil moisture content change. Consider the use of belled caissons with supported floor. Open block wall foundations have been utilized for small houses. Collect rainwater falling on structure and surrounding areas and convey runoff away from structures.
Construction of light to medium load structures in hot, arid climate where the free surface evaporation is several times larger than annual rainfall. Difficulties are greatest in fractured and slickensided clay of high activity, with low water table and maximum deficiency of evaporation over rainfall. (South Africa, as an example.)	Permanent moisture deficiency exists in the ground. Construction eliminates evaporation over building area, reducing capillary stresses and causing movement of moisture to beneath building. This leads to continuing heave with minor seasonal fluctuations. Thermoosmotic gradients directed toward cooled subsoil beneath structure contribute to increase in moisture, which may extend to depths of 10 to 15 ft.	Damage is minimized by use of slab or raft foundation, dry wall construction, steel or reinforced concrete framing, reinforced foundation beams, and provision for jacking. Heave is eliminated by removal of desiccated material to a depth of 8 to 12 ft.and replacement by granular fill; or belled caissons, founded near the water table and reinforced to resist tensile forces, supporting floor between caissons with opening or compressible filler beneath floors. Divert rainwater and surface runoff away from structure.
Chemical changes:		
Excavation and exposure of clay-shales or shales containing pyrite (iron sulphide) or anhydrite (calcium sulphate).	Exposure to air and water causes oxidation and hydration of pyrites with a volumetric expansion of as much as ten times their original volume, or hydration of anhydrite to gypsum.	Rough excavate no closer than one-half foot to final subgrade and protect exposed shale with a spray or mop coat of bitumen. When ready for foundations, excavate to final grade and pour concrete immediately over a spray or mop coat of bitumen.



MATERIALS INVESTIGATED ARE CLAYS, HIGHLY OVERCONSOLIDATED BY CAPILLARY STRESSES THAT ARE EFFECTIVE PRIOR TO THE CONSTRUCTION OF THE STRUCTURE UPON THEM.

PROCEDURE FOR ESTIMATING TOTAL SWELL UNDER STRUCTURE LOAD.

1. OBTAIN REPRESENTATIVE UNDISTURBED SAMPLES OF THE SHALLOW CLAY STRATUM AT A TIME WHEN CAPILLARY STRESSES ARE EFFECTIVE; I.E., WHEN NOT FLOODED OR SUBJECTED TO HEAVY RAIN.
2. LOAD SPECIMENS (AT NATURAL WATER CONTENT) IN CONSOLIDOMETER UNDER A PRESSURE EQUAL TO THE ULTIMATE VALUE OF OVERBURDEN FOR HIGH GROUND WATER, PLUS WEIGHT OF STRUCTURE. ADD WATER TO SATURATE AND MEASURE SWELL.
3. COMPUTE FINAL SWELL IN TERMS OF PERCENT OF ORIGINAL SAMPLE HEIGHT AND PLOT SWELL VERSUS DEPTH, AS IN THE LEFT PANEL.
4. COMPUTE TOTAL SWELL WHICH IS EQUAL TO THE AREA UNDER THE PERCENT SWELL VERSUS DEPTH CURVE. FOR THE ABOVE EXAMPLE:

   TOTAL SWELL = $1/2\ (8.2-1.0) \times 2.8/100 = 0.10$ FT.

PROCEDURE FOR ESTIMATING UNDERCUT NECESSARY TO REDUCE SWELL TO AN ALLOWABLE VALUE.

1. FROM PERCENT SWELL VERSUS DEPTH CURVE PLOT RELATIONSHIP OF TOTAL SWELL VERSUS DEPTH AT THE RIGHT. TOTAL SWELL AT ANY DEPTH EQUALS AREA UNDER THE CURVE AT LEFT, INTEGRATED UPWARD FROM THE DEPTH OF ZERO SWELL.
2. FOR A GIVEN ALLOWABLE VALUE OF SWELL, READ THE AMOUNT OF UNDERCUT NECESSARY FROM THE TOTAL SWELL VERSUS DEPTH CURVE. FOR EXAMPLE, FOR AN ALLOWABLE SWELL OF 0.03 FT, UNDERCUT REQUIRED = 4.6 FT. UNDERCUT CLAY IS REPLACED BY AN EQUAL THICKNESS OF NONSWELLING COMPACTED FILL.

FIGURE 22
Computation of Swell of Desiccated Clays

REFERENCES

1. Department of Civil Engineering, University of California, Berkeley, Stresses and Deflections in Foundations and Pavements, Fall, 1965.

2. Duncan, J.M., and Buchignani, A.L., An Engineering Manual For Settlement Studies, University of California, Berkeley, 1976.

3. D'Appolonia, D.J., Poulos, H.G. and Ladd, C.C., Initial Settlement of Structures on Clay, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 97, No. SM10, 1971.

4. Schmertmann, J.H., Static Cone to Compute Static Settlement Over Sand, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM3, 1976.

5. Oweis, I.S., Equivalent Linear Model For Predicting Settlements of Sand Bases, Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT12, 1979.

6. Leonards, G.S., Estimating Consolidation Settlements of Shallow Foundation on Overconsolidated Clay, Transportation Research Board Special Report 163, Transportation Research Board, 1976.

7. Departments of the Army and Air Force, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures, (Except Hydraulic Structures), TM5-818-1/AFM-88-3, Chapter 7, Washington, D.C., 1979.

8. Terzaghi, K. and Peck, R., Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, 1967.

9. Ladd, C.C., Foott, R., Ishihara, K., Schlosser, F., and Poulos, H.G., Stress Deformation and Strength Characteristics, Proceedings Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Volume 2, pp 421-494, 1977.

10. Olson, R.E., Consolidation Under Time Dependent Loading, Journal of the Geotechnical Engineering Division, ASCE, Vol. 103, No. GT1, 1977.

11. Institution of Civil Engineers, Structure Soil Interaction, A State of the Art Report, 1978.

12. Bjerrum, L., Allowable Settlements of Structures, Proceedings of European Conference on Soil Mechanics and Foundation Engineering, Wiesbaden, Volume 2, pp 135-137, 1963.

13. Brown, P.T., Numerical Analyses of Uniformly Loaded Circular Rafts on Elastic Layers of Finite Depth, Geotechnique, Vol. 19, No. 2, 1969.

14. Frazer, R.A. and Wardle, L.J., Numerical Analysis of Rectangular Raft on Layered Foundations, Geotechnique, Vol. 26, No. 4, 1976.

15. Ladd, C.C., Use of Precompression and Vertical Sand Drains for Stabilization of Foundation Soils, ASCE New York Metropolitan Section Spuinar, 1978.

CHAPTER 6. SEEPAGE AND DRAINAGE

Section 1. INTRODUCTION

1. SCOPE. This chapter covers surface erosion, and analysis of flow quantity and groundwater pressures associated with underseepage. Requirements are given for methods of drainage and pressure relief.

2. RELATED CRITERIA. Other criteria, relating to groundwater utilization or control, can be found in the following sources:

Subject	Source
Drainage Systems	NAVFAC DM-5.03
Drainage for Airfield Pavements	NAVFAC DM-21.06
Dewatering and Groundwater Control for Deep Excavations..	NAVFAC P-418

Additional criteria for permanent pressure relief and seepage control beneath structures are given in DM-7.02, Chapter 4.

3. APPLICATIONS. Control of soil erosion must be considered in all new construction projects. Seepage pressures are of primary importance in stability analysis and in foundation design and construction. Frequently, drawdown of groundwater is necessary for construction. In other situations, pressure relief must be incorporated in temporary and permanent structures.

4. INVESTIGATIONS REQUIRED. For erosion analysis, the surface water flow characteristics, soil type, and slope are needed. For analysis of major seepage problems, determine permeability and piezometric levels by field observations. See Chapter 2 for techniques.

Section 2. SEEPAGE ANALYSIS

1. FLOW NET. Figure 1 shows an example of flow net construction. Use this procedure to estimate seepage quantity and distribution of pore water pressures in two-dimensional flow. Flow nets are applicable for the study of cutoff walls and wellpoints, or shallow drainage installations placed in a rectangular layout whose length in plan is several times its width. Flow nets can also be used to evaluate concentration of flow lines.

a. Groundwater Pressures. For steady state flow, water pressures depend on the ratio of mean permeability of separate strata and the anisotropy of layers. A carefully drawn flow net is necessary to determine piezometric levels within the flow field or position of the drawdown curve.

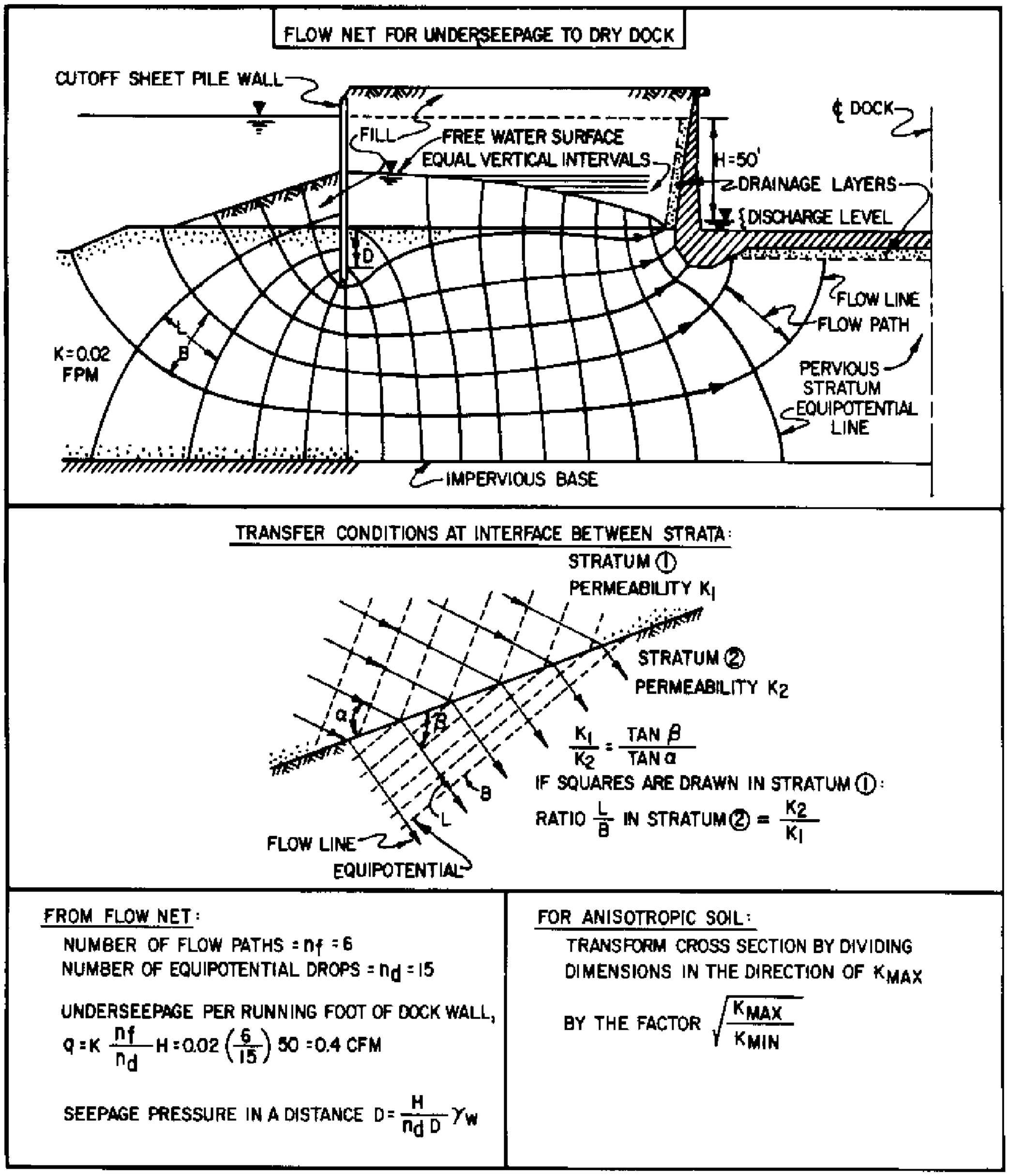


FIGURE 1
Flow Net Construction and Seepage Analysis

))
RULES FOR FLOW NET CONSTRUCTION
)))))))))))))))))))))))))))))))

1. WHEN MATERIALS ARE ISOTROPIC WITH RESPECT TO PERMEABILITY, THE PATTERN OF FLOW LINES AND EQUIPOTENTIALS INTERSECT AT RIGHT ANGLES. DRAW A PATTERN IN WHICH SQUARE FIGURES ARE FORMED BETWEEN FLOW LINES AND EQUIPOTENTIALS

2. USUALLY IT IS EXPEDIENT TO START WITH AN INTEGER NUMBER OF EQUIPOTENTIAL DROPS, DIVIDING TOTAL HEAD BY A WHOLE NUMBER, AND DRAWING FLOW LINES TO CONFORM TO THESE EQUIPOTENTIALS. IN THE GENERAL CASE, THE OUTER FLOW PATH WILL FORM RECTANGULAR RATHER THEN SQUARE FIGURES. THE SHAPE OF THESE RECTANGLES (RATIO B/L) MUST BE CONSTANT.

3. THE UPPER BOUNDARY OF A FLOW NET THAT IS AT ATMOSPHERIC PRESSURE IS A "FREE WATER SURFACE". INTEGER EQUIPOTENTIALS INTERSECT THE FREE WATER SURFACE AT POINTS SPACED AT EQUAL VERTICAL INTERVALS.

4. A DISCHARGE FACE THROUGH WHICH SEEPAGE PASSES IS AN EQUIPOTENTIAL LINE IF THE DISCHARGE IS SUBMERGED, OR A FREE WATER SURFACE IF THE DISCHARGE IS NOT SUBMERGED. IF IT IS A FREE WATER SURFACE, THE FLOW NET FIGURES ADJOINING THE DISCHARGE FACE WILL NOT BE SQUARES.

5. IN A STRATIFIED SOIL PROFILE WHERE RATIO OF PERMEABILITY OF LAYERS EXCEEDS 10, THE FLOW IN THE MORE PERMEABLE LAYER CONTROLS. THAT IS, THE FLOW NET MAY BE DRAWN FOR MORE PERMEABLE LAYER ASSUMING THE LESS PERMEABLE LAYER TO BE IMPERVIOUS. THE HEAD ON THE INTERFACE THUS OBTAINED IS IMPOSED ON THE LESS PERVIOUS LAYER FOR CONSTRUCTION OF THE FLOW NET WITHIN IT.

6. IN A STRATIFIED SOIL PROFILE WHERE RATIO OF PERMEABILITY OF LAYERS IS LESS THAN 10, FLOW IS DEFLECTED AT THE INTERFACE IN ACCORDANCE WITH THE DIAGRAM SHOWN ABOVE.

7. WHEN MATERIALS ARE ANISOTROPIC WITH RESPECT TO PERMEABILITY, THE CROSS SECTION MAY BE TRANSFORMED BY CHANGING SCALE AS SHOWN ABOVE AND FLOW NET DRAWN AS FOR ISOTROPIC MATERIALS. IN COMPUTING QUANTITY OF SEEPAGE, THE DIFFERENTIAL HEAD IS NOT ALTERED FOR THE TRANSFORMATION.

8. WHERE ONLY THE QUANTITY OF SEEPAGE IS TO BE DETERMINED, AN APPROXIMATE FLOW NET SUFFICES. IF PORE PRESSURES ARE TO BE DETERMINED, THE FLOW NET MUST BE ACCURATE.
))

FIGURE 1 (continued)

Flow Net Construction and Seepage Analysis

b. Seepage Quantity. Total seepage computed from flow net depends primarily on differential head and mean permeability of the most pervious layer. The ratio of permeabilities of separate strata or their anisotropy has less influence. The ratio n_f/n_d in Figure 1 usually ranges from 1/2 to 2/3 and thus for estimating seepage quantity a roughly drawn flow net provides a reasonably accurate estimate of total flow. Uncertainties in the permeability values are much greater limitations on accuracy.

For special cases, the flow regime can be analyzed by the finite element method. Mathematical expressions for the flow are written for each of the elements, considering boundary conditions. The resulting system of equations is solved by computer to obtain the flow pattern (see Appendix A).

2. SEEPAGE FORCES. The flow of water through soil exerts a force on the soil called a seepage force. The seepage pressure is this force per unit volume of soil and is equal to the hydraulic gradient times the unit weight of water.

$$P_s = i\,\gamma_w$$

where

P_s = seepage pressure

i = hydraulic gradient

γ_w = unit weight of water

The seepage pressure acts in a direction at right angles to the equipotential lines (see Figure 1).

The seepage pressure is of great importance in analysis of the stability of excavations and slopes (see Chapter 7 and DM-7.2, Chapter 1) because it is responsible for the phenomenon known as boiling or piping.

a. Boiling. Boiling occurs when seepage pressures in an upward direction exceed the downward force of the soil. The condition can be expressed in terms of critical hydraulic gradient. A minimum factor of safety of 2 is usually required, i.e.,

$$i_c = i_{CRITICAL} = \frac{\gamma_T - \gamma_W}{\gamma_W} = \frac{\gamma_b}{\gamma_W} \;;$$

$$F_s = \frac{i_c}{i} = 2$$

where

i = actual hydraulic gradient

γ_T = total unit weight of the soil

γ_W = unit weight of water

γ_b = buoyant unit weight of soil

b. Piping and Subsurface Erosion. Most piping failures are caused by subsurface erosion in or beneath dams. These failures can occur several months or even years after a dam is placed into operation.

In essence, water that comes out of the ground at the toe starts a process of erosion (if the exit gradient is high enough) that culminates in the formation of a tunnel-shaped passage (or "pipe") beneath the structure. When the passage finally works backward to meet the free water, a mixture of soil and water rushes through the passage, undermining the structure and flooding the channel below the dam. It has been shown that the danger of a piping failure due to subsurface erosion increases with decreasing grain size.

Similar subsurface erosion problems can occur in relieved drydocks, where water is seeping from a free source to a drainage or filter blanket beneath the floor or behind the walls. If the filter fails or is defective and the hydraulic gradients are critical, serious concentrations of flow can result in large voids and eroded channels.

Potential passageways for the initiation of piping include: uniformly graded gravel deposits, conglomerate, open joints in bedrock, cracks caused by earthquakes or crustal movements, open joints in pipelines, hydraulic fracture, open voids in coarse boulder drains including French drains, abandoned wellpoint holes, gopher holes, cavities formed in levee foundations by rotting roots or buried wood, improper backfilling of pipelines, pipes without antiseepage collars, etc.

Failure by piping requires progressive movement of soil particles to a free exit surface. It can be controlled by adequately designed filters or relief blankets. Guidelines for preventing piping beneath dams may be found in Reference 1, <u>Security from Under Seepage of Masonary Dams on Earth Foundations</u>, by Lee.

3. DEWATERING. Dewatering methods are discussed in Table 7, DM-7.2, Chapter 1. Figures 13 and 14 in DM-7.2, Chapter 1 illustrate some methods of construction dewatering and soil grain size limitations for different dewatering methods. See NAVFAC P-418 for dewatering and groundwater control systems.

4. THREE-DIMENSIONAL FLOW. For analysis of flow quantity and drawdown to individual wells or to any array of wells, see Section 5.

Section 3. SEEPAGE CONTROL BY CUTOFF

1. METHODS. Procedures for seepage control include cutoff walls for decreasing the seepage quantity and reducing the exit gradients, and drainage or relief structures that increase flow quantity but reduce seepage pressures or cause drawdown in critical areas. See Table 1; Table 7 of DM-7.2, Chapter 1; and DM-7.3, Chapter 3 (Diaphragm Walls) for methods of creating partial or complete cutoff. See NAVFAC P-418 for construction dewatering.

2. SHEETPILING. A driven line of interlocking steel sheeting may be utilized for a cutoff as a construction expedient or as a part of the completed structure.

a. <u>Applicability</u>. The following considerations govern the use of sheetpiling:

TABLE 1
Cutoff Methods for Seepage Control

Method	Applicability	Characteristics and Requirements
Sheet pile cut off wall	Suited especially for stratified soils with high horizontal and low vertical permeability or pervious hydraulic fill materials. May be easily damaged by boulders or buried obstructions. Tongue and groove wood sheeting utilized for shallow excavation in soft to medium soils. Interlocking steel sheetpiling is utilized for deeper cutoff.	Steel sheeting must be carefully driven to maintain interlocks tight. Steel H-pile soldier beams may be used to minimize deviation of sheeting in driving. Some deviation of sheeting from plumb toward the side with least horizontal pressure should be expected. Seepage through interlocks is minimized where tensile force acts across interlocks. For straight wall sheeting an appreciable flow may pass through interlocks. Decrease interlock leakage by filling interlocks with sawdust, bentonite, cement grout, or similar material.
Compacted barrier of impervious soil	Formed by compacted backfill in a cutoff trench carried down to impervious material or as a core section in earth dams.	Layers or streaks of pervious material in the impervious zone must be avoided by careful selection and mixing of borrow materials, scarifying lifts, aided by sheepsfoot rolling. A drainage zone downstream of an impervious section of the embankment is necessary in most instances.

TABLE 1 (continued)
Cutoff Methods for Seepage Control

Method	Applicability	Characteristics and Requirements
Grouted or injected cutoff	Applicable where depth or character of foundation materials make sheetpile wall or cutoff trench impractical. Utilized extensively in major hydraulic structures. May be used as a supplement below cutoff sheeting or trenches.	A complete positive grouted cutoff is often difficult and costly to attain, requiring a pattern of holes staggered in rows with carefully planned injection sequence and pressure control. See DM-7.3, Chapter 2 for materials and methods.
Slurry trench method	Suited for construction of impervious cutoff trench below groundwater or for stabilizing trench excavation. Applicable whenever cutoff walls in earth are required. Is replacing sheetpile cutoff walls.	Vertical sided trench is excavated below groundwater as slurry with specific gravity generally between 1.2 and 1.8 is pumped back into the trench. Slurry may be formed by mixture of powdered bentonite with fine-grained material removed from the excavation. For a permanent cutoff trench, such as a foundation wall or other diaphragm wall, concrete is tremied to bottom of trench, displacing slurry upward. Alternatively, well graded backfill material is dropped through the slurry in the trench to form a dense mixture that is essentially an incompressible mixture; in working with coarser gravels (which may settle out), to obtain a more reliable key into rock, and a narrower trench, use a cement-bentonite mix.

TABLE 1 (continued)
Cutoff Methods for Seepage Control

Method	Applicability	Characteristics and Requirements
Impervious wall of mixed in-place piles.	Method may be suitable to form cofferdam wall where sheet pile cofferdam is expensive or cannot be driven to suitable depths, or has insufficient rigidity, or requires excessive bracing.	For a cofferdam surrounding an excavation, a line of overlapping mixed in-place piles are formed by a hollow shaft auger or mixing head rotated into the soil while cement grout is pumped through the shaft. Where piles cannot be advanced because of obstructions or boulders, supplementary grouting or injection may be necassary.
Freezing - ammonium brine or liquid nitrogen	All types of saturated soils and rock. Forms ice in voids to stop water. Ammonium brine is better for large applications of long duration. Liquid nitrogen is better for small applications of short duration where quick freezing is needed.	Gives temporary mechanical strength to soil. Installation costs are high and refrigeration plant is expensive. Some ground heave occurs.
See also DM-7.2 Chapter 1, Table 10, DM-7.3 Chapter 3 (for diaphragm walls as a cutoff), and DM-7.3 Chapter 2 (for grouted cutoffs and freezing).		

(1) Sheeting is particularly suitable in coarse-grained material with maximum sizes less than about 6 inches or in stratified subsoils with alternating fine grained and pervious layers where horizontal permeability greatly exceeds vertical.

(2) To be effective, sheeting must be carefully driven with interlocks intact. Boulders or buried obstructions are almost certain to damage sheeting and break interlock connections. Watertightness cannot be assumed if obstructions are present.

(3) Loss of head across a straight wall of intact sheeting depends on its watertightness relative to the permeability of the surrounding soil. In homogeneous fine-grained soil, head loss created by sheeting may be insignificant. In pervious sand and gravel, head loss may be substantial depending on the extent to which the flow path is lengthened by sheeting. In this case, the quantity of water passing through intact interlocks may be as much as 0.1 gpm per foot of wall length for each 10 feet differential in head across sheeting, unless special measures are taken to seal interlocks.

b. Penetration Required. This paragraph and Paragraph "c" below apply equally to all impervious walls listed in Table 1. Seepage beneath sheeting driven for partial cutoff may produce piping in dense sands or heave in loose sands. Heave occurs if the uplift force at the sheeting toe exceeds the submerged weight of the overlying soil column. To prevent piping or heave of an excavation carried below groundwater, sheeting must penetrate a sufficient depth below subgrade or supplementary drainage will be required at subgrade. See Figure 2 (Reference 2, Model Experiments to Study the Influence of Seepage on the Stability of a Sheeted Excavation in Sand, by Marsland) for sheeting penetration required for various safety factors against heave or piping in isotropic sands. For homogeneous but anisotropic sands, reduce the horizontal cross-section dimensions by the transformation factor of Figure 1 to obtain the equivalent cross section for isotropic conditions. See Figure 3 (Reference 2) for sheeting penetration required in layered subsoils. For clean sand, exit gradients between 0.5 and 0.75 will cause unstable conditions for men and equipment operating on the subgrade. To avoid this, provide sheeting penetration for a safety factor of 1.5 to 2 against piping or heave.

c. Supplementary Measures. If it is uneconomical or impractical to provide required sheeting penetration, the seepage exit gradients may be reduced as follows:

(1) For homogeneous materials or soils whose permeability decreases with depth, place wellpoints, pumping wells, or sumps within the excavation. Wellpoints and pumping wells outside the excavation are as effective in some cases and do not interfere with bracing or excavation.

(2) For materials whose permeability increases with depth, ordinary relief wells with collector pipes at subgrade may suffice.

(3) A pervious berm placed against the sheeting, or a filter blanket at subgrade, will provide weight to balance uplift pressures. Material placed directly on the subgrade should meet filter criteria of Section 4.

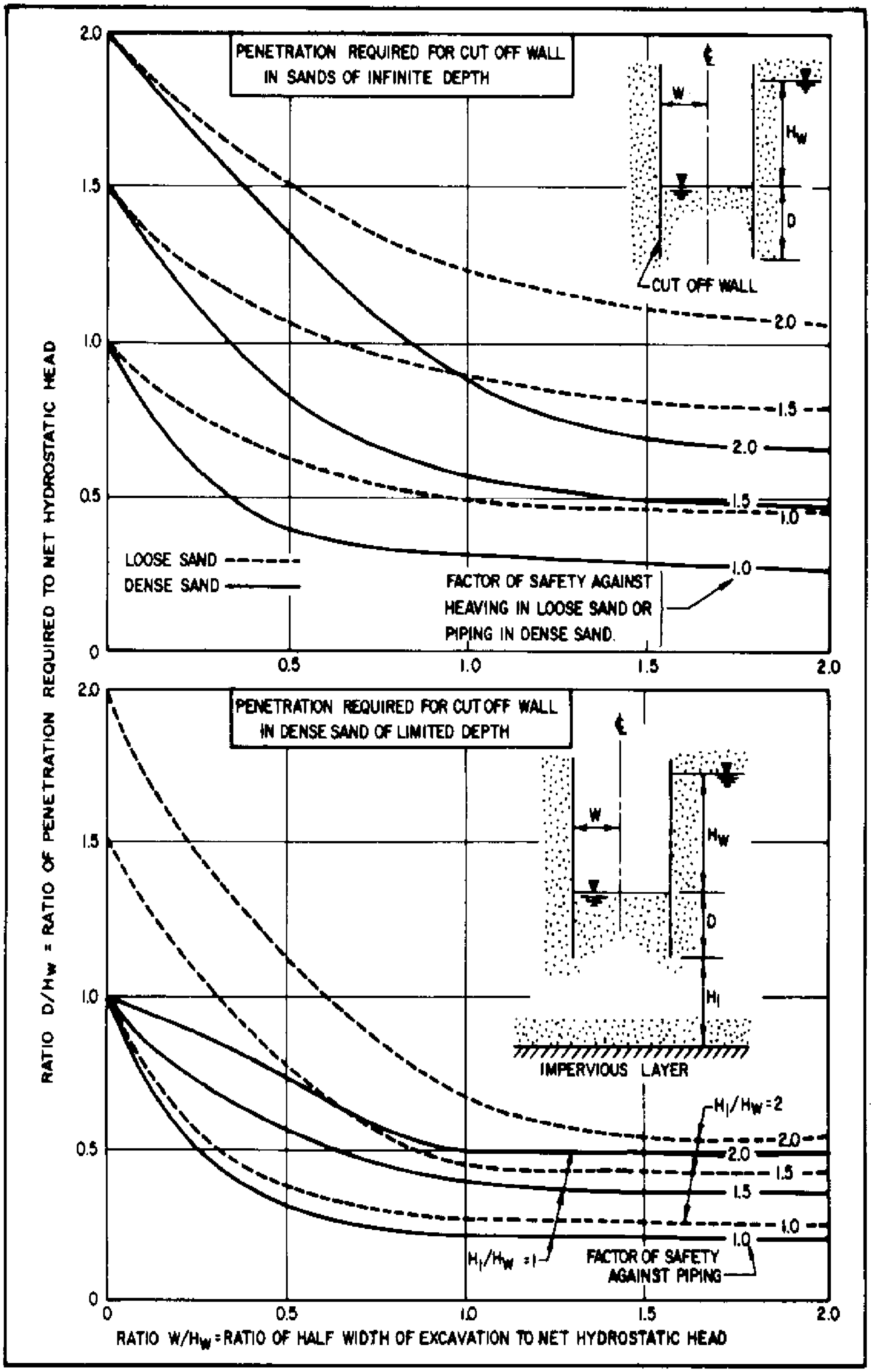
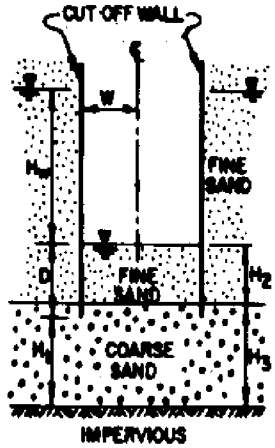


FIGURE 2
Penetration of Cut Off Wall to Prevent Piping in Isotropic Sand

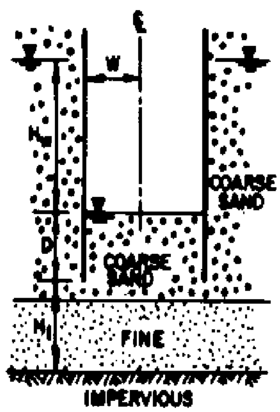


COARSE SAND UNDERLYING FINE SAND

PRESENCE OF COARSE LAYER MAKES FLOW IN FINE MATERIAL MORE NEARLY VERTICAL AND GENERALLY INCREASES SEEPAGE GRADIENTS IN THE FINE LAYER COMPARED TO THE HOMOGENEOUS CROSS-SECTION OF FIGURE 2.

IF TOP OF COARSE LAYER IS AT A DEPTH BELOW CUT OFF WALL BOTTOM GREATER THAN WIDTH OF EXCAVATION, SAFETY FACTORS OF FIGURE 2 FOR INFINITE DEPTH APPLY.

IF TOP OF COARSE LAYER IS AT A DEPTH BELOW CUT OFF WALL BOTTOM LESS THAN WIDTH OF EXCAVATION, THE UPLIFT PRESSURES ARE GREATER THAN FOR THE HOMOGENEOUS CROSS-SECTION. IF PERMEABILITY OF COARSE LAYER IS MORE THAN TEN TIMES THAT OF FINE LAYER, FAILURE HEAD (H_W) = THICKNESS OF FINE LAYER (H_2).



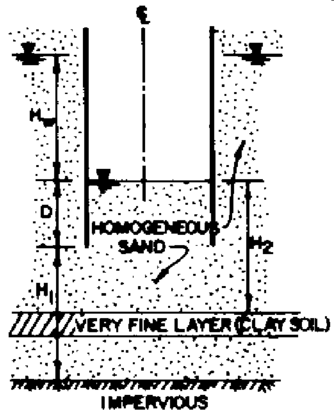
FINE SAND UNDERLYING COARSE SAND

PRESENCE OF FINE LAYER CONSTRICTS FLOW BENEATH CUT OFF WALL AND GENERALLY DECREASES SEEPAGE GRADIENTS IN THE COARSE LAYER.

IF TOP OF FINE LAYER LIES BELOW CUT OFF WALL BOTTOM, SAFETY FACTORS ARE INTERMEDIATE BETWEEN THOSE FOR AN IMPERMEABLE BOUNDARY AT TOP OR BOTTOM OF THE FINE LAYER USING FIGURE 2.

IF TOP OF THE FINE LAYER LIES ABOVE CUT OFF WALL BOTTOM, THE SAFETY FACTORS OF FIGURE 2 ARE SOMEWHAT CONSERVATIVE FOR PENETRATION REQUIRED.

FIGURE 3
Penetration of Cut Off Wall Required to Prevent Piping in Stratified Sand

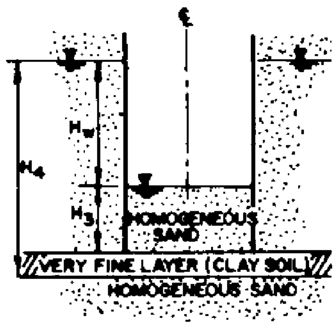


FINE LAYER IN HOMOGENEOUS SAND STRATUM

IF THE TOP OF FINE LAYER IS AT A DEPTH GREATER THAN WIDTH OF EXCAVATION BELOW CUT OFF WALL BOTTOM, SAFETY FACTORS OF FIGURE 2 APPLY, ASSUMING IMPERVIOUS BASE AT TOP OF FINE LAYER.

IF TOP OF FINE LAYER IS AT A DEPTH LESS THAN WIDTH OF EXCAVATION BELOW CUT OFF WALL TIPS, PRESSURE RELIEF IS REQUIRED SO THAT UNBALANCED HEAD BELOW FINE LAYER DOES NOT EXCEED HEIGHT OF SOIL ABOVE BASE OF LAYER.

IF FINE LAYER LIES ABOVE SUBGRADE OF EXCAVATION, FINAL CONDITION IS SAFER THAN HOMOGENEOUS CASE, BUT DANGEROUS CONDITION MAY ARISE DURING EXCAVATION ABOVE THE FINE LAYER AND PRESSURE RELIEF IS REQUIRED AS IN THE PRECEDING CASE.



TO AVOID BOTTOM HEAVE, $\gamma_T \times H_3$ SHOULD BE GREATER THAN $\gamma_W \times H_4$.

γ_T = TOTAL UNIT WEIGHT OF THE SOIL

γ_W = UNIT WEIGHT OF WATER

FIGURE 3 (continued)
Penetration of Cut Off Wall Required to Prevent Piping in Stratified Sand

(4) An outside open water source may be blanketed with fines or bentonite dumped through water or placed as a slurry. See Table 2.

Evaluate the effectiveness of these measures by flow net analysis.

3. GROUTED CUTOFF. For grouting methods and materials, see DM-7.3, Chapter 2. Complete grouted cutoff is frequently difficult and costly to attain. Success of grouting requires careful evaluation of pervious strata for selection of appropriate grout mix and procedures. These techniques, in combination with other cutoff or drainage methods, are particularly useful as a construction expedient to control local seepage.

4. IMPERVIOUS SOIL BARRIERS. Backfilling of cutoff trenches with selected impervious material and placing impervious fills for embankment cores are routine procedures for earth dams.

a. Compacted Impervious Fill. Properly constructed, these sections permit negligible seepage compared to the flow through foundations or abutments. Pervious layers or lenses in the compacted cutoff must be avoided by blending of borrow materials and scarifying to bond successive lifts.

b. Mixed-in-Place Piles. Overlapping mixed-in-place piles of cement and natural soil forms a cofferdam with some shear resistance around an excavation.

c. Slurry-filled Trench. Concurrent excavation of a straight sided trench and backfilling with a slurry of bentonite with natural soil is done. Alternatively, a cement bentonite mix can be used in a narrower trench where coarser gravel occurs. In certain cases, tremie concrete may be placed, working upward from the base of a slurry-filled trench, to form a permanent peripheral wall (Diaphragm Wall, see DM-7.3, Chapter 3).

5. FREEZING. See Section 2, DM-7.3, Chapter 2, and Table 7, DM-7.2, Chapter 1.

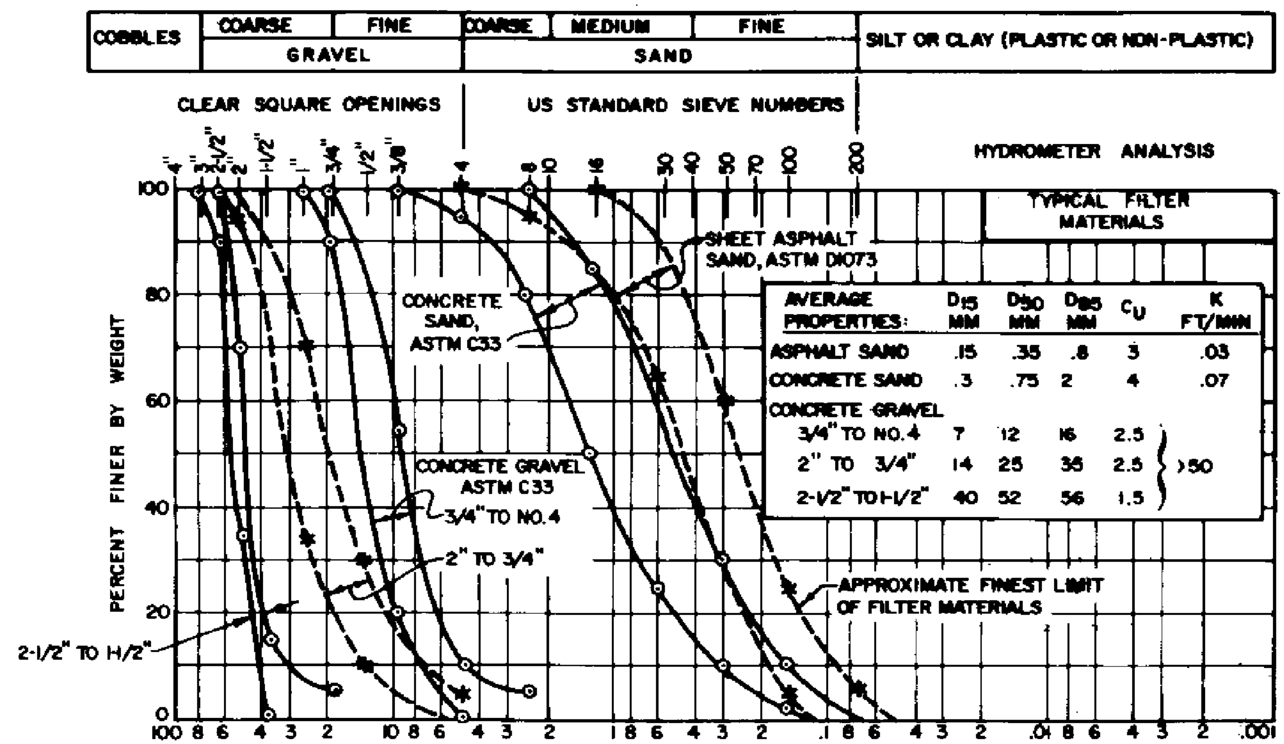
Section 4. DESIGN OF DRAINAGE BLANKET AND FILTERS

1. FILTERS. If water flows from a silt to a gravel, the silt will wash into the interstices of the gravel. This could lead to the following, which must be avoided:

(1) The loss of silt may continue, causing creation of a cavity.

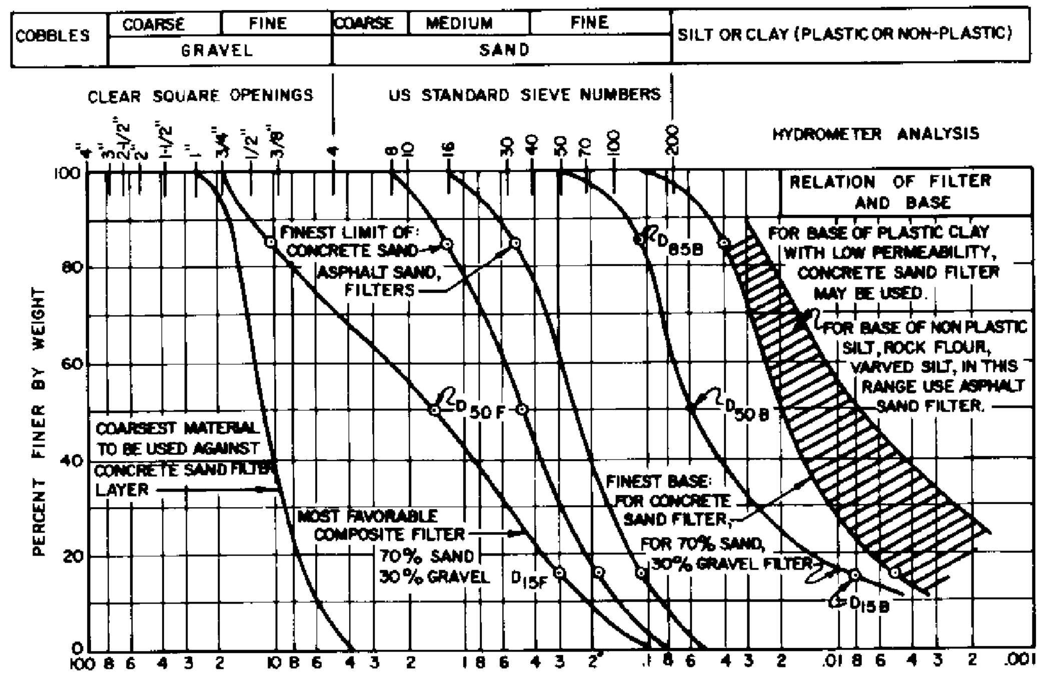
(2) The silt may clog the gravel, stopping flow, and causing hydrostatic pressure buildup.

The purpose of filters is to allow water to pass freely across the interface (filter must be coarse enough to avoid head loss) but still be sufficiently fine to prevent the migration of fines. The filter particles must be durable, e.g., certain crushed limestones may dissolve. Filter requirements apply to all permanent subdrainage structures in contact with soil, including wells. See Figure 4 for protective filter design criteria.



AVERAGE PROPERTIES:	D_{15} MM	D_{50} MM	D_{85} MM	C_U	K FT/MIN
ASPHALT SAND	.15	.35	.8	3	.03
CONCRETE SAND	.3	.75	2	4	.07
CONCRETE GRAVEL					
3/4" TO NO. 4	7	12	16	2.5	>50
2" TO 3/4"	14	25	35	2.5	>50
2-1/2" TO 1-1/2"	40	52	56	1.5	>50

FIGURE 4
Design Criteria for Protective Filters



GENERAL REQUIREMENTS:

1. TO AVOID HEAD LOSS IN FILTER: $\frac{D_{15F}}{D_{15B}} > 4$, AND PERMEABILITY OF FILTER MUST BE LARGE ENOUGH TO SUFFICE FOR PARTICULAR DRAINAGE SYSTEM.
2. TO AVOID MOVEMENT OF PARTICLES FROM BASE: $\frac{D_{15F}}{D_{85B}} < 5$, $\frac{D_{50F}}{D_{50B}} < 25$, $\frac{D_{15F}}{D_{15B}} < 20$

   FOR VERY UNIFORM BASE MATERIAL ($C_u < 1.5$): D_{15F}/D_{85B} MAY BE INCREASED TO 6

   FOR BROADLY GRADED BASE MATERIAL ($C_u > 4$): D_{15F}/D_{15B} MAY BE INCREASED TO 40
3. TO AVOID MOVEMENT OF FILTER IN DRAIN PIPE PERFORATIONS OR JOINTS: D_{85F}/SLOT WIDTH > (1.2 TO 1.4), D_{85F}/HOLE DIAMETER > (1.0 TO 1.2)
4. TO AVOID SEGREGATION, FILTER SHOULD CONTAIN NO SIZES LARGE THAN 3".
5. TO AVOID INTERNAL MOVEMENT OF FINES, FILTER SHOULD HAVE NO MORE THAN 5% PASSING No. 200 SIEVE.

FIGURE 4 (continued)
Design Criteria for Protective Filters

The filter may be too fine grained to convey enough water, to provide a good working surface, or to pass the water freely without loss of fines to a subdrain pipe. For this condition, a second filter layer is placed on the first filter layer; the first filter layer is then considered the soil to be protected, and the second filter layer is designed. The finest filter soil is often at the base, with coarser layers above. This is referred to as reversed or inverted filters.

Concrete sand (ASTM C33, Specifications for Concrete Aggregates) suffices as a filter against the majority of fine-grained soils or silty or clayey sands. For non-plastic silt, varved silt, or clay with sand or silt lenses, use asphalt sand (ASTM D1073, Specifications for Fine Aggregates for Bituminous Paving Mixtures) but always check the criteria in Figure 4. Locally available natural materials are usually more economical than processed materials, and should be used where they meet filter criteria. The fine filter layer can be replaced with plastic filter cloths under the following conditions (after Reference 3, Performance of Plastic Filter Cloths as a Replacement for Granular Materials, by Calhoun, et al.):

(a) Non-woven filter cloths, or woven filter cloths with less than 4% open area should not be used where silt is present in sandy soils. A cloth with an equivalent opening size (EOS) equal to the No. 30 sieve and an open area of 36% will retain sands containing silt.

(b) When stones are to be dropped directly on the cloth, or where uplift pressure from artesian water may be encountered, the minimum tensile strengths (ASTM D1682, Tests for Breaking Load and Elongation of Textile Fabrics) in the strongest and weakest directions should be not less than 350 and 200 lbs. respectively. Elongation at failure should not exceed 35%. The minimum burst strength should be 520 psi (ASTM D751, Testing Coated Fabrics). Where the cloths are used in applications not requiring high strength or abrasion resistance, the strength requirements may be relaxed.

(c) Cloths made of polypropylene, polyvinyl chloride and polyethylene fibers do not deteriorate under most conditions, but they are affected by sunlight, and should be protected from the sun. Materials should be durable against ground pollutants and insect attack, and penetration by burrowing animals.

(d) Where filter cloths are used to wrap collection pipes or in similar applications, backfill should consist of clean sands or gravels graded such that the D+85, is greater than the EOS of the cloth. When trenches are lined with filter cloth, the collection pipe should be separated from the cloth by at least six inches of granular material.

(e) Cloths should be made of monofilament yarns, and the absorption of the cloth should not exceed 1% to reduce possibility of fibers swelling and changing EOS and percent of open area.

For further guidance on types and properties of filter fabrics see Reference 4, Construction and Geotechnical Engineering Using Synthetic Fabrics, by Koerner and Welsh.

2. DRAINAGE BLANKET. Figure 5 shows typical filter and drainage blanket installations.

a. Permeability. Figure 6 (Reference 5, Subsurface Drainage of Highways, by Barber) gives typical coefficients of permeability for clean, coarse-grained drainage material and the effect of various percentages of fines on permeability. Mixtures of about equal parts gravel with medium to coarse sand have a permeability of approximately 1 fpm. Single sized, clean gravel has a permeability exceeding 50 fpm. For approximate relationship of permeability versus effective grain size D+10,, see Figure 1, Chapter 3.

b. Drainage Capacity. Estimate the quantity of water which can be transmitted by a drainage blanket as follows:

q = k [multiplied by] i [multiplied by] A

where

q = quantity of flow, ft.3-/sec

k = permeability coefficient, ft/sec

i = average gradient in flow direction, ft/ft

A = cross sectional area of blanket, ft.2-

The gradient is limited by uplift pressures that may be tolerated at the point farthest from the outlet of the drainage blanket. Increase gradients and flow capacity of the blanket by providing closer spacing of drain pipes within the blanket.

(1) Pressure Relief. See bottom panel of Figure 7 (Reference 6, Seepage Requirements of Filters and Pervious Bases, by Cedergren) for combinations of drain pipe spacing, drainage course thickness, and permeability required for control of flow upward from an underlying aquifer under an average vertical gradient of 0.4.

(2) Rate of Drainage. See the top panel of Figure 7 (Reference 5) for time rate of drainage of water from a saturated base course beneath a pavement. Effective porosity is the volume of drainable water in a unit volume of soil. It ranges from 25 percent for a uniform material such as medium to coarse sand, to 15 percent for a broadly graded sand-gravel mixture.

c. Drainage Blanket Design. The following guidelines should be followed:

(1) Gradation. Design in accordance with Figure 4.

(2) Thickness. Beneath, structures require a minimum of 12 inches for each layer with a minimum thickness of 24 inches overall. If placed on wet, yielding, uneven excavation surface and subject to construction operation and traffic, minimum thickness shall be 36 inches overall.

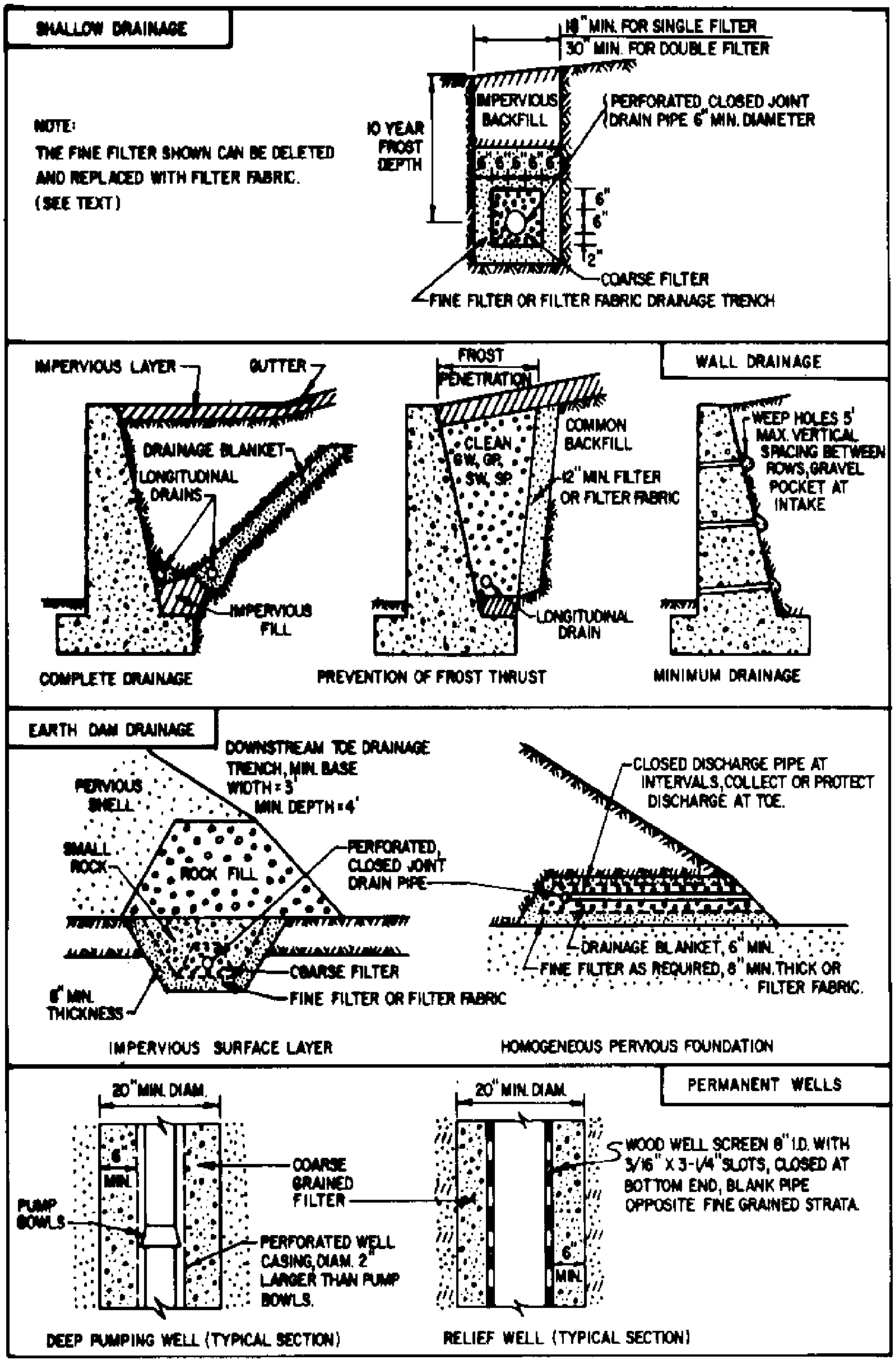


FIGURE 5
Typical Filter and Drainage Blanket Applications

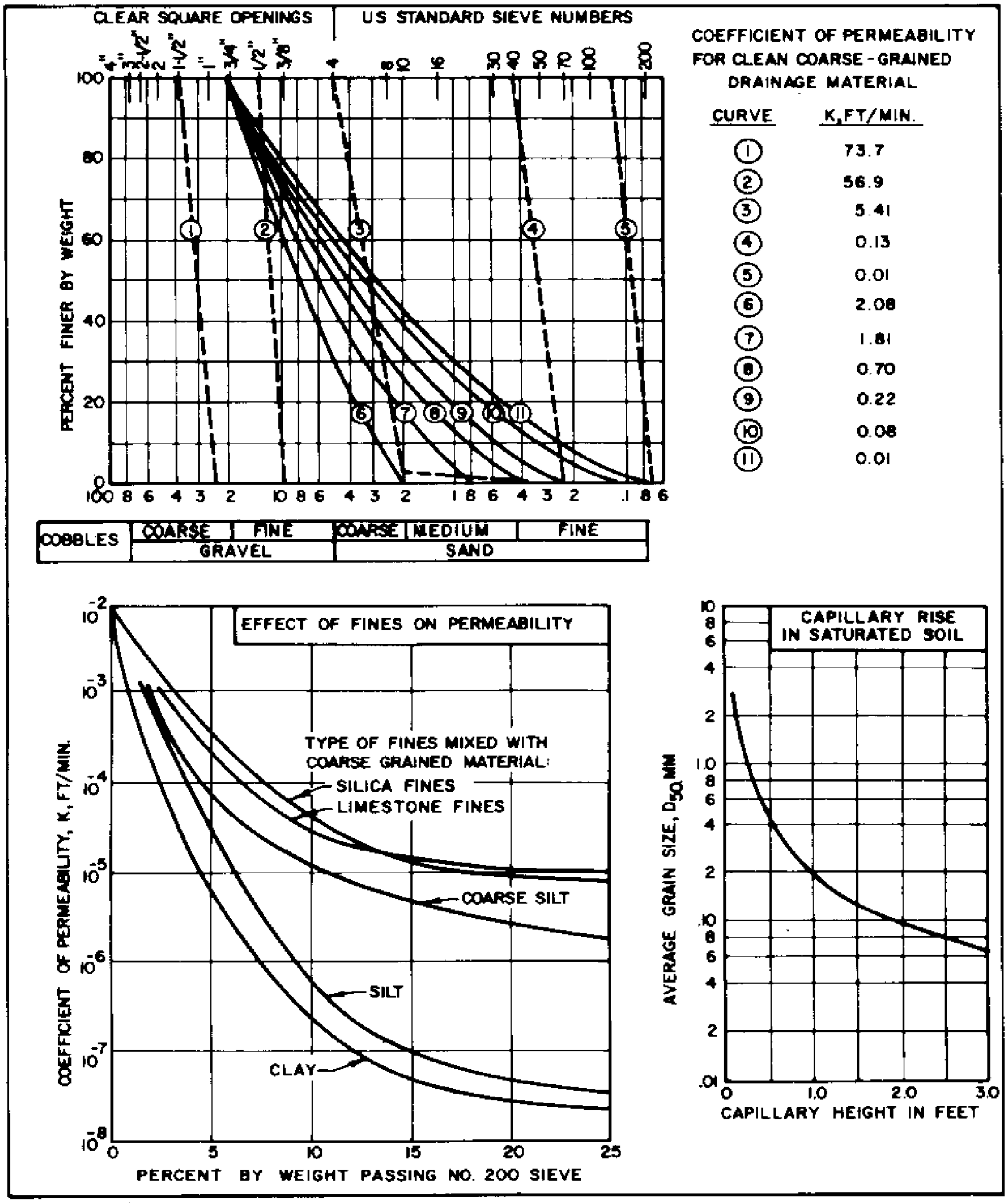


FIGURE 6
Permeability and Capillarity of Drainage Materials

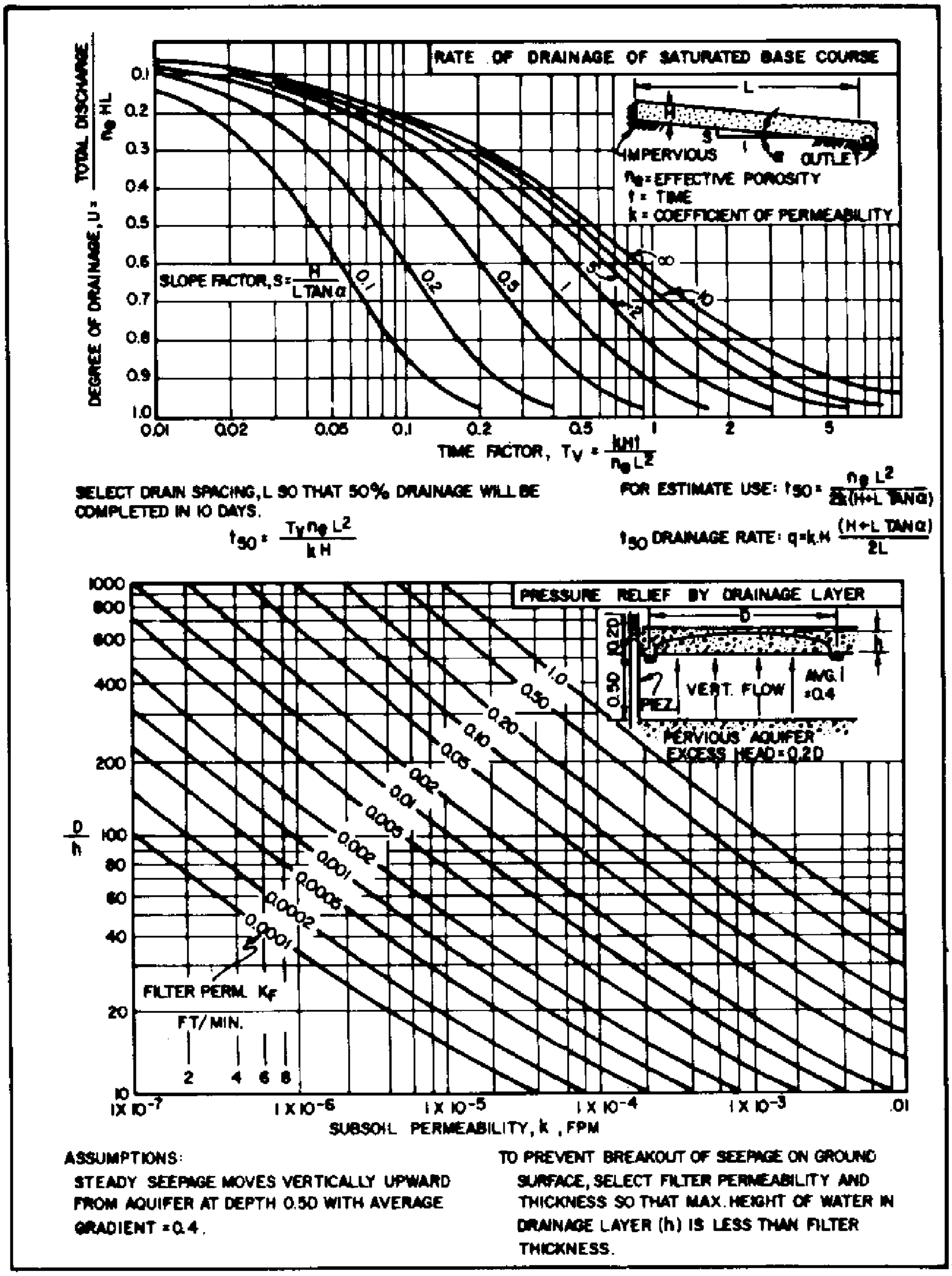


FIGURE 7
Analysis of Drainage Layer Performance

d. Chemical Clogging. Filter systems (filter layers, fabrics, pipes) can become chemically clogged by ferruginous (iron) and carbonate depositions and incrustations. Where the permanent subdrainage system is accessible, pipes with larger perforations (3/8 inch) and increased thickness of filter layers can be used. For existing facilities, a weak solution of hydrochloric acid can be used to dissolve carbonates.

3. INTERCEPTING DRAINS. Intercepting drains consist of shallow trenches with collector pipes surrounded by drainage material, placed to intercept seepage moving horizontally in an upper pervious stratum. To design proper control drains, determine the drawdown and flow to drains by flow net analysis. Figure 8 shows typical placements of intercepting drains for roadways on a slope.

4. SHALLOW DRAINS FOR PONDED AREAS. Drains consisting of shallow stone trenches with collector pipes can be used to collect and control surface runoff. See Figure 9 (Reference 7, Seepage Into Ditches From a Plane Water Table Overlying a Gravel Substratum, by Kirkham; and Reference 8, Seepage Into Ditches in the Case of a Plane Water Table And an Impervious Substratum, by Kirkham) for determination of rate of seepage into drainage trenches. If sufficient capacity cannot be provided in trenches, add surface drainage facilities.

5. PIPES FOR DRAINAGE BLANKETS AND FILTERS. Normally, perforated wall pipes of metal or plastic or porous wall concrete pipes are used as collector pipes. Circular perforations should generally not be larger than 3/8 inch. Filter material must be graded according to the above guidelines.

Pipes should be checked for strength. Certain deep buried pipes may need a cradle. Check for corrosiveness of soil and water; certain metal pipes may not be appropriate.

Since soil migration may occur, even in the best designed systems, install cleanout points so that the entire system can be flushed and snaked.

Section 5. WELLPOINT SYSTEMS AND DEEP WELLS

1. METHODS. Excavation below groundwater in soils having a permeability greater than 10^{-3} fpm generally requires dewatering to permit construction in the dry. For materials with a permeability between 10^{-3} and 10^{-5} fpm, the amount of seepage may be small but piezometric levels may need to be lowered in order to stabilize slopes or to prevent softening of subgrades. Drawdown for intermediate depths is normally accomplished by wellpoint systems or sumps.

Deep drainage methods include deep pumping wells, relief wells, and deep sheeted sumps. These are appropriate when excavation exceeds a depth that can be dewatered efficiently by wellpoint systems alone or when the principal source of seepage is from lower permeable strata.

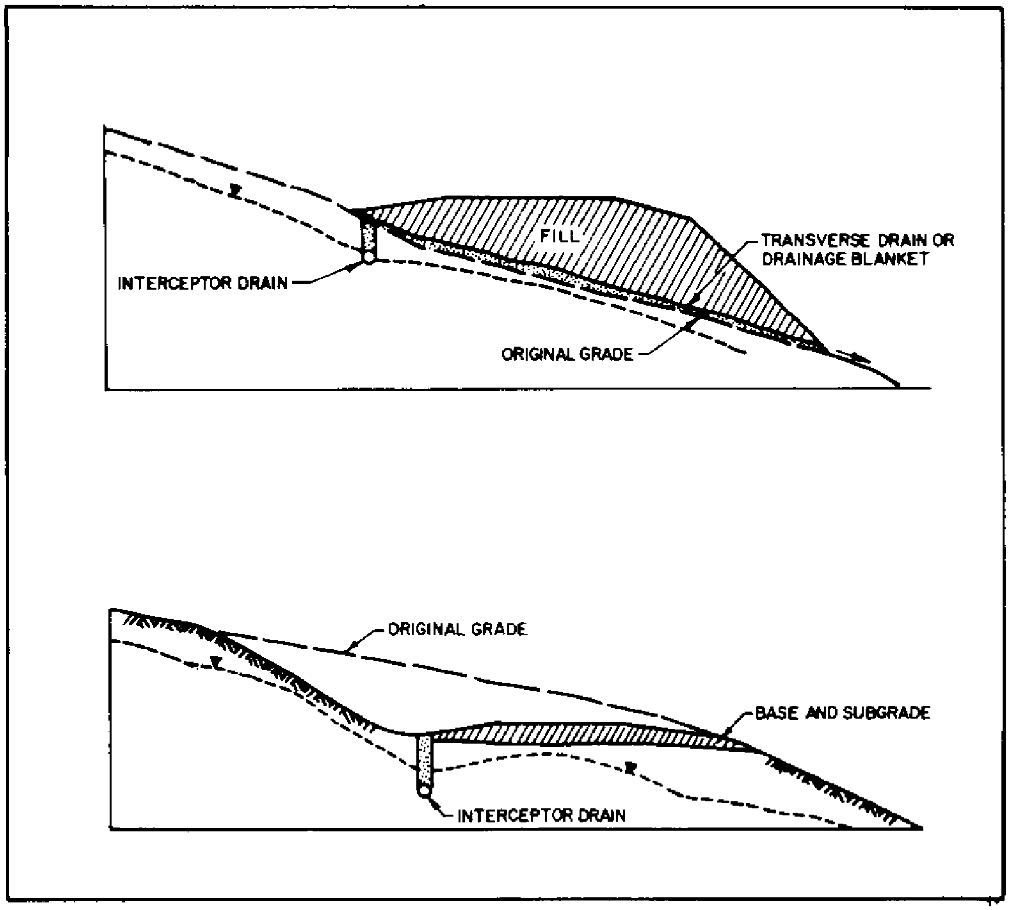


FIGURE 8
Intercepting Drains for Roadways on a Slope

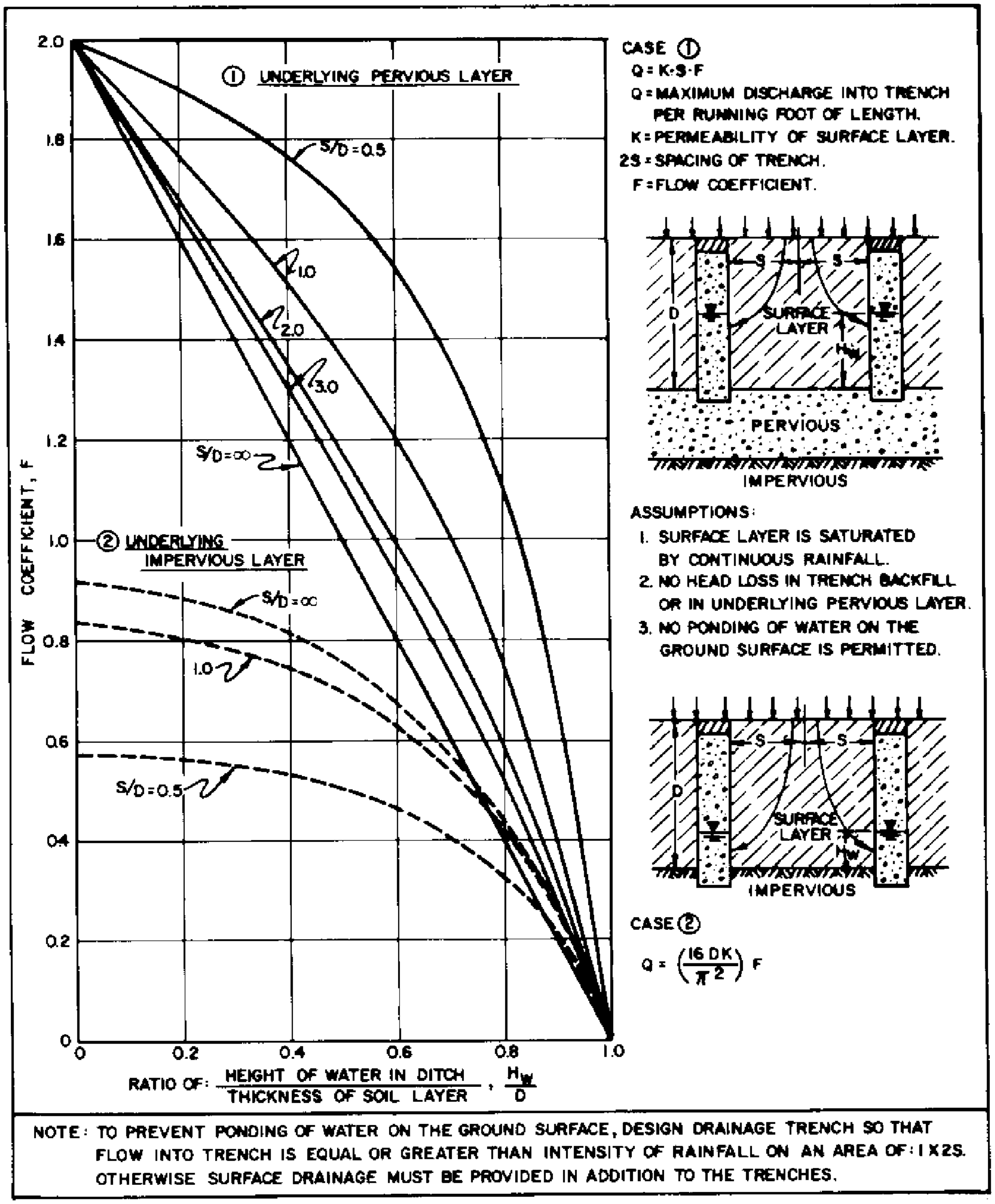


FIGURE 9
Rate of Seepage into Drainage Trench

a. Construction Controls. For important construction dewatering, install piezometers below the base of excavations and behind slopes or cofferdams to check on the performance and adequacy of drainage system.

b. Settlement Effects. Where dewatering lowers the water levels in permeable strata adjacent to compressible soils, settlement may result. See Chapter 5 for methods of settlement evaluation.

c. Dewatering Schemes. For construction of dewatering systems and procedures, refer to DM-7.2, Chapter 1, and NAVFAC P-418.

2. WELLPOINT SYSTEMS. Wellpoints consist of 1-1/2 or 2-inch diameter pipes with a perforated bottom section protected by screens. They are jetted or placed in a prepared hole and connected by a header pipe to suction pumps.

a. Applicability. Wellpoints depend upon the water flowing by gravity to the well screen. Pumping methods for gravity drainage generally are not effective when the average effective grain size of a soil D+10, is less than 0.05 mm. In varved or laminated soils where silty fine sands are separated by clayey silts or clay, gravity drainage may be effective even if the average material has as much as 50 percent smaller than 0.05 mm. Compressible, fine-grained materials containing an effective grain size less than 0.01 mm can be drained by providing a vacuum seal at the ground surface around the wellpoint, utilizing atmospheric pressure as a consolidating force. See Section 4 for limitations due to iron and carbonate clogging.

b. Capacity. Wellpoints ordinarily produce a drawdown between 15 and 18 feet below the center of the header. For greater drawdown, install wellpoints in successive tiers or stages as excavation proceeds. Discharge capacity is generally 15 to 30 gpm per point. Points are spaced between 3 and 10 feet apart. In finely stratified or varved materials, use minimum spacing of points and increase their effectiveness by placing sand in the annular space surrounding the wellpoint.

c. Analysis. Wellpoint spacing usually is so close that the seepage pattern is essentially two dimensional. Analyze total flow and drawdown by flow net procedure. (See Section 2.) For fine sands and coarser material, the quantity of water to be removed controls wellpoint layout. For silty soils, the quantity pumped is relatively small and the number and spacing of wellpoints will be influenced by the time available to accomplish the necessary drawdown.

3. SUMPS. For construction convenience or to handle a large flow in pervious soils, sumps can be excavated with soldier beam and horizontal wood lagging. Collected seepage is removed with centrifugal pumps placed within the sump. Analyze drawdown and flow quantities by approximating the sump with an equivalent circular well of large diameter.

Sheeted sumps are infrequently used. Unsheeted sumps are far more common, and are used primarily in dewatering open shallow excavations in coarse sands, clean gravels, and rock.

4. ELECTRO-OSMOSIS. This is a specialized procedure utilized in silts and clays that are too fine-grained to be effectively drained by gravity or vacuum methods. See DM-7.03, Chapter 2.

5. PUMPING WELLS. These wells are formed by drilling a hole of sufficient diameter to accommodate a pipe column and filter, installing a well casing, and placing filter material in the annular space surrounding the casing. Pumps may be either the turbine type with a motor at the surface and pipe column with pump bowls hung inside the well, or a submersible pump placed within the well casing.

a. Applications. Deep pumping wells are used if (a) dewatering installations must be kept outside the excavation area, (b) large quantities are to be pumped for the full construction period, and (c) pumping must commence before excavation to obtain the necessary time for drawdown. See Figure 10 (bottom panel, Reference 9, Analysis of Groundwater Lowering Adjacent to Open Water, by Avery) for analysis of drawdown and pumping quantities for single wells or a group of wells in a circular pattern. Deep wells may be used for gravels to silty fine sands, and water bearing rocks. See Section 4 for limitations due to iron and carbonate clogging.

Bored shallow wells with suction pumps can be used to replace wellpoints where pumping is required for several months or in silty soils where correct filtering is critical.

b. Special Methods. Ejector or eductor pumps may be utilized within wellpoints for lifts up to about 60 feet. The ejector pump has a nozzle arrangement at the bottom of two small diameter riser pipes which remove water by the Venturi principle. They are used in lieu of a multistage wellpoint system and if the large pumping capacity of deep wells is not required. Their primary application is for sands, but with proper control they can also be used in silty sands and sandy silts.

6. RELIEF WELLS. These wells are sand columns used to bleed water from underlying strata containing artesian pressures, and to reduce uplift forces at critical location. Relief wells may be tapped below ground by a collector system to reduce back pressures acting in the well.

a. Applications. Relief wells are frequently used as construction expedients, and in situations where a horizontal drainage course may be inadequate for pressure relief of deep foundations underlain by varved or stratified soils or soils whose permeability increases with depth.

b. Analysis. See Figure 11 (Reference 10, Soil Mechanics Design, Seepage Control, by the Corps of Engineers) for analysis of drawdown produced by line of relief wells inboard of a long dike. To reduce uplift pressures h+m, midway between the wells to safe values, vary the well diameter, spacing, and penetration to obtain the best combination.

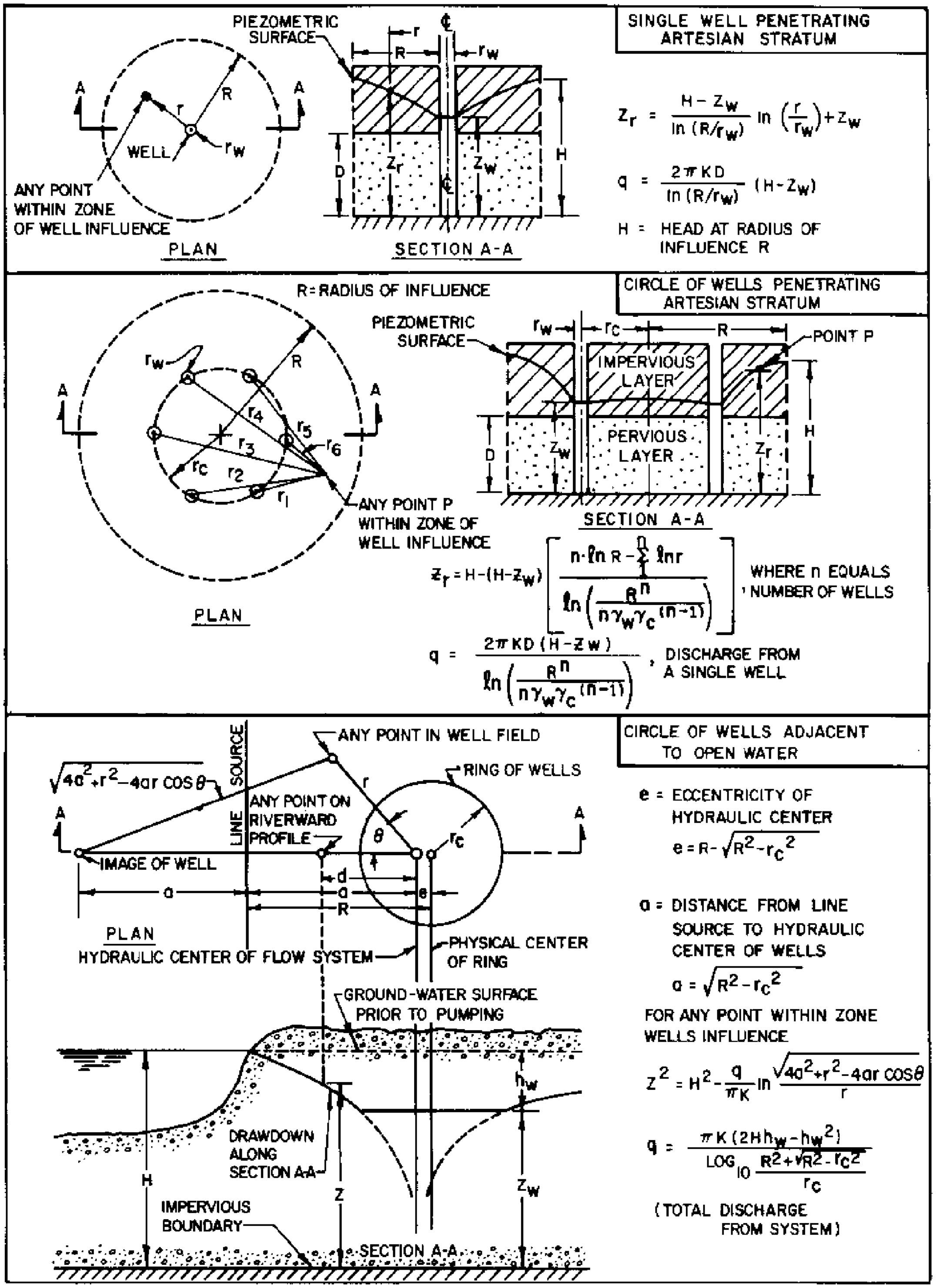


FIGURE 10
Groundwater Lowering by Pumping Wells

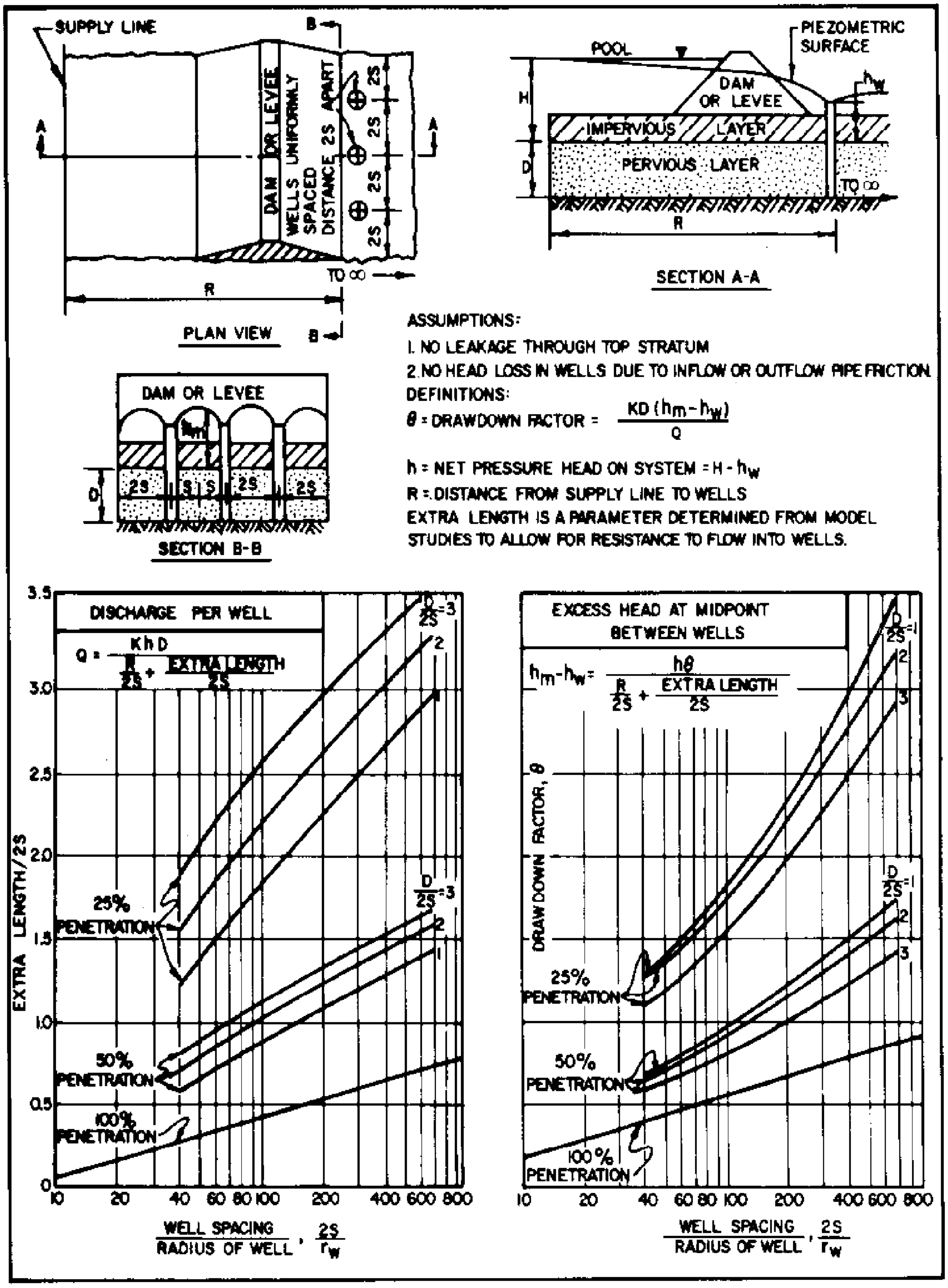


FIGURE 11
Drainage of Artesian Layer by Line of Relief Wells

Section 6. LININGS FOR RESERVOIRS AND POLLUTION CONTROL FACILITIES

1. PURPOSE. Linings are used to reduce water loss, to minimize seepage which can cause instability in embankments, and to keep pollutants from migrating to groundwater sources as in holding ponds at sewage treatment and chemical facilities, and in sanitary landfills. For further guidance see Reference 4 and Reference 11, Wastewater Stabilization Pond Linings, by the Cold Regions Research and Engineering Laboratory.

2. TYPES. Table 2 lists types of linings appropriate where wave forces are insignificant. Where erosive forces are present, combine lining with slope protection procedure. See Chapter 7, Section 6.

3. SUBDRAINAGE. If the water level in the reservoir may fall below the surrounding groundwater level, a permanent subdrainage system should be provided below the lining.

4. INVESTIGATION FOR LINING. Check any potential lining for reaction to pollutants (e.g., synthetic rubber is subject to attack by hydrocarbons), potential for insect attack (e.g., certain synthetic fabrics may be subject to termite attack), and the potential for borrowing animals breaching the lining.

Section 7. EROSION CONTROL

1. GENERAL. The design of erosion controls must consider the volume of runoff from precipitation, the runoff velocity, and the amount of soil loss.

a. Volume of Runoff. The volume of runoff depends on the amount of precipitation, ground cover, and topography. For guidance on evaluating the volume of runoff see DM-5.3 or Reference 12, Urban Hydrology for Small Watersheds, by the Soil Conservation Service.

b. Amount of Soil Loss. Soil losses can be estimated using the Universal Soil Loss Equation developed by the Soil Conservation Service:

$$A = EI \text{ [multiplied by] } KLS$$

where

A = computed soil loss per acre, in tons

EI = rainfall erosion index

K = soil erodibility factor

L = slope length factor

S = slope gradient factor

TABLE 2
Impermeable Reservoir Linings

Method	Applicability and Procedures
Buried Plastic Liner	Impervious liner formed of black colored polyvinyl chloride plastic film. Where foundation is rough or rocky, place a layer 2 to 4 inches thick of fine-grained soil beneath liner. Seal liner sections by bonding with manufacturer's recommended solvent with 6-inch overlap at joints. Protect liner by 6-inch min. cover of fine grained soil. On slopes add a 6-inch layer of gravel and cobbles 3/4 to 3-inch size. Anchor liner in a trench at top of slope. Avoid direct contact with sunlight during construction before covering with fill and in completed installation. Usual thickness range of 20 to 45 mils (.020" to 045"). Items to be specified include Tensile Strength (ASTM D412), Elongation at Break (ASTM D412), Water Absorption (ASTM D471), Cold Bend (ASTM D2136), Brittleness Temperature (ASTM D746), Ozone Resistance (ASTM D1149), Heat Aging Tensile Strength and Elongation at Break (ASTM D412), Strength - Tear and Grab (ASTM D751).
Buried Synthetic Rubber Liner	Impervious liner formed by synthetic rubber, most often polyester reinforced. Preparation, sealing, protection, anchoring, sunlight, thickness, and ASTM standards are same as Buried Plastic Liner.
Bentonite Seal	Bentonite placed under water to seal leaks after reservoir filling. For placing under water, bentonite may be poured as a powder or mixed as a slurry and placed into the reservoir utilizing methods recommended by the manufacturer. Use at least 0.8 pounds of bentonite for each square foot of area, with greater concentration at location of suspected leaks. For sealing silty or sandy soils, bentonite should have no more than 10 percent larger than 0.05 mm; for gravelly and rocky materials, bentonite can have as much as 40 percent larger than 0.05 mm. For sealing channels with flowing water or large leaks, use mixture of 1/3 each of sodium bentonite, calcium bentonite, and sawdust.

TABLE 2 (continued)
Impermeable Reservoir Linings

Method	Applicability and Procedures
Earth Lining	Lining generally 2 to 4 feet thick of soils having low permeability. Used on bottom and sides of reservoir extending to slightly above operating water levels. Permeability of soil should be no greater than about $2x10^{-6}$ fpm for water supply linings and $2x10^{-7}$ fpm for pollution control facility linings.
Thin Compacted Soil Lining with Chemical Dispersant	Dispersant is utilized to minimize thickness of earth lining required by decreasing permeability of the lining. Used where wave action is not liable to erode the lining. Dispersant, such as sodium tetraphosphate, is spread on a 6-inch lift of clayey silt or clayey sand. Typical rate of application is 0.05 lbs/sf. Chemical and soil are mixed with a mechanical mixer and compacted by sheepsfoot roller. Using a suitable dispersant, the thickness of compacted linings may be limited to about 1 foot; the permeability of the compacted soil can be reduced to 1/10 of its original value.

EI, L, and S values should be obtained from local offices of the U.S. Soil Conservation Service. K values may be determined from published data ona particular locality. In the absence of such data, it may be roughly estimated from Figure 12 (after Reference 13, Erosion Control on Highway Construction, by the Highway Research Board).

2. INVESTIGATION. Where erosion can be expected during earthwork construction, on-site investigations should include: (1) field identification and classification for both agricultural textures and the Unified system, (2) sampling for grain size distribution, Atterberg limits and laboratory classification, and (3) determination of in-place densities (see Chapter 2).

3. SURFACE EROSION CONTROL. For typical erosion control practices see Table 3, (modified from Reference 13). General considerations to reduce erosion include:

a. Construction Scheduling. Schedule construction to avoid seasons of heavy rains. Winds are also seasonal, but are negligible in impact compared to water erosion.

b. Soil Type. Avoid or minimize exposure of highly erodible soils. Sands easily erode but are easy to trap. Clays are more erosion resistant, but once eroded, are more difficult to trap.

c. Slope Length and Steepness. Reduce slope lengths and steepness to reduce velocities. Provide benches on slopes at maximum vertical intervals of 30 feet.

d. Cover. Cover quickly with vegetation, such as grass, shrubs and trees, or other covers such as mulches. A straw mulch applied at 2 tons/acre may reduce soil losses as much as 98% on gentle slopes. Other mulches include asphalt emulsion, paper products, jute, cloth, straw, wood chips, sawdust, netting of various natural and man-made fibers, and, in some cases, gravel.

e. Soil Surface. Ridges perpendicular to flow and loose soil provide greater infiltration.

f. Exposed Area. Minimize the area opened at any one time. Retain as much natural vegetation as possible. Leave vegetation along perimeters to control erosion and act as a sediment trap.

g. Diversion. Minimize flow over disturbed areas, such as by placing a berm at the top of a disturbed slope.

h. Sprinkling. Control dust by sprinkling of exposed areas.

i. Sediment Basins. Construct debris basins to trap debris and silt before it enters streams.

4. CHANNEL LININGS. Table 4 presents guidelines for minimizing erosion of earth channels and grass covered channels (modified after Reference 14, Minimizing Erosion in Urbanizing Areas, by the Soil Conservation Service).

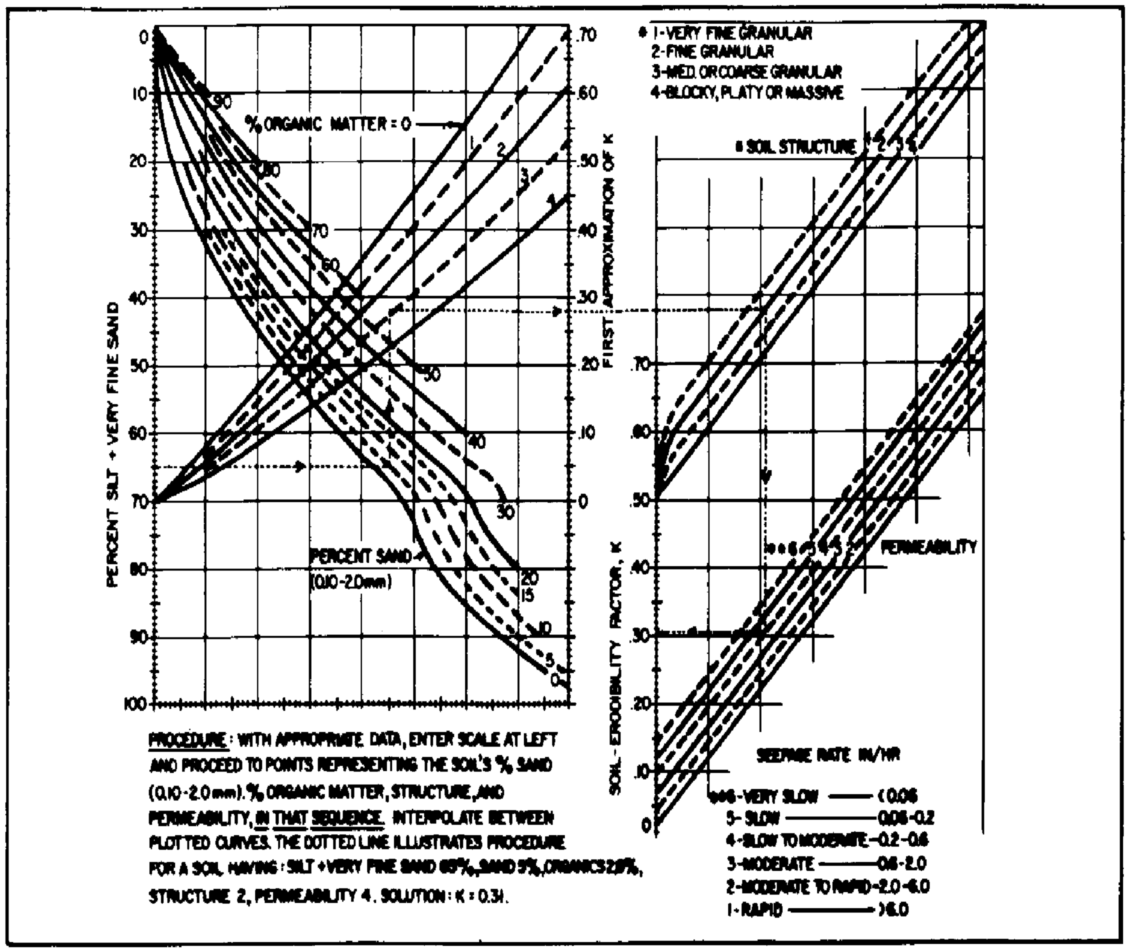


FIGURE 12
Nomograph for Determining Soil Erodibility (K) for
Universal Soil Loss Equation

TABLE 3
Typical Erosion Control Practice

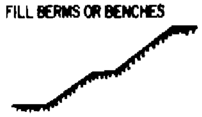
Treatment Practice	Advantages	Problems
FILL SLOPES		
BERMS AT TOP OF EMBANKMENT 	Prevent runoff from embankment surface from flowing over face of fill Collect runoff for slope drains or protected ditch Can be placed as a part of the normal construction operation and incorporated into fill or shoulders	Cooperation of construction operators to place final lifts at edge or shaping into berm Difficult to compact outside lift when work is resumed Sediment buildup and berm and slope failure
SLOPE DRAINS 	Prevent fill slope erosion caused by embankment surface runoff Can be constructed of full or half section pipe, bituminous, metal, concrete, plastic, or other waterproof material Can be extended as construction progresses May be either temporary or permanent	Permanent construction as needed may not be considered desirable by contractor Removal of temporary drains may disturb growing vegetation Energy dissipation devices are required at the outlets
FILL BERMS OR BENCHES 	Slows velocity of slope runoff Collects sediments Provides access for maintenance Collects water for slope drains May utilize waste	Requires additional fill material if waste is not available May cause sloughing Additional construction area may be needed
SEEDING/MULCHING 	Timely application of mulch and seeding decreases the period a slope is subject to severe erosion Mulch that is cut in or otherwise anchored will collect sediment. The furrows made will also hold water and sediment	Seeding season may not be favorable Not 100 percent effective in preventing erosion Watering may be necessary Steep slopes or locations with high velocities may require supplemental treatment.

TABLE 3 (continued)
Typical Erosion Control Practice

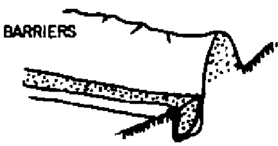
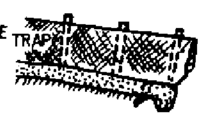
Treatment Practice	Advantages	Problems
PROTECTION OF ADJACENT PROPERTY		
BRUSH BARRIERS 	Use slashing and logs from clearing operation Can be covered and seeded rather than removed Eliminates need for burning or disposal off right-of-way	May be considered unsightly in urban areas
STRAW BALE BARRIERS 	Straw is readily available in many areas When properly installed, they filter sediment and some turbidity from runoff	Requires removal Subject to vandal damage Flow is slow through straw requiring considerable area
SEDIMENT TRAPS 	Collect much of the sediment spill from fill slopes and storm drain ditches Inexpensive Can be cleaned and expanded to meet need	Does not eliminate all sediment and turbidity Space is not always available
SEDIMENT POOLS 	Can be designed to handle large volumes of flow Both sediment and turbidity are removed May be incorporated into permanent erosion control plan	Requires prior planning, additional construction area and/or flow easement If removal is necessary, can present a major effort during final construction stage Clean-out volumes can be large Access for clean-out not always convenient; Anti-seepage baffles required for permanent construction
FENCE TRAP 	Low cost Temporary measure can be erected with minimum supervision	Some maintenance needed depending on length of time in place

TABLE 3 (continued)
Typical Erosion Control Practice

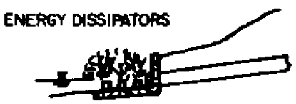
Treatment Practice	Advantages	Problems
PROTECTION OF ADJACENT PROPERTY (continued)		
ENERGY DISSIPATORS 	Slow velocity to permit sediment collection and to minimize channel erosion off project	Collects debris and requires cleaning Requires special design and construction of large shot rock or other suitable material from project
LEVEL SPREADERS 	Spreads channel or pipe flow to sheet flow Avoids channel easements and construction off project Simple to construct	Adequate spreader length may not be available Sodding of overflow berm is usually required Must be a part of the permanent erosion control effort Maintenance forces must maintain spreader until no longer required
PROTECTION OF STREAM		
CONSTRUCTION DIKE 	Permits work to continue during normal stream stages Controlled flooding can be accomplished during periods of inactivity	Usually requires pumping of work site water into sediment pond Subject to erosion from stream and from direct rainfall on dike
COFFERDAM 	Work can be continued during most anticipated stream conditions Clear water can be pumped directly back into stream No material deposited in stream	Expensive
TEMPORARY STREAM CHANNEL CHANGE	Prepared channel keeps normal flows away from construction	New channel usually will require protection Stream must be returned to old channel and temporary channel refilled

TABLE 3 (continued)
Typical Erosion Control Practice

Treatment Practice	Advantages	Problems
PROTECTION OF STREAM (continued)		
RIP RAP 	Sacked sand with cement or stone easy to stockpile and place Can be installed in increments as needed	Expensive
TEMPORARY CULVERTS FOR HAUL ROADS 	Eliminates stream turbulence and turbidity Provides unobstructed passage for fish and other aquatic life Capacity for normal flow can be provided with storm water flowing over the roadway	Space not always available without conflicting with permanent structure work May be expensive, especially for larger sizes of pipe Subject to washout
ROCK-LINED LOW-LEVEL CROSSING 	Minimizes stream turbidity Inexpensive May also serve as ditch check or sediment trap	May not be fordable during rainstorms During periods of low flow, passage of fish may be blocked

TABLE 3 (continued)
Typical Erosion Control Practice

Treatment Practice	Advantages	Problems
DITCHES		
CHECK DAMS 	Maintains low velocities Catches sediment Can be constructed of logs, shot rock, lumber, masonry or concrete, gabions, sand bags	Close spacing on steep grades Require clean-out Unless keyed at sides and bottom, erosion may occur
SEDIMENT TRAPS/ STRAW BALE FILTERS 	Can be located as necessary to collect sediment during construction Clean-out often can be done with on-the-job equipment Simple to construct	Little direction on spacing and size Sediment disposal may be difficult Specification must include provisions for periodic clean-out May require seeding, sodding or pavement when removed during final cleanup
SODDING 	Easy to place with a minimum of preparation Can be repaired during construction Immediate protection May be used on sides of paved ditches to provide increased capacity	Requires water during first few weeks Sod not always available Will not withstand high velocity or severe abrasion from sediment load
SEEDING WITH MULCH AND MATTING 	Usually least expensive Effective for ditches with low velocity Easily placed in small quantities with inexperienced personnel	Will not withstand medium to high velocity Requires anchoring
PAVING, RIPRAP, RUBBLE	Effective for high velocities May be part of the permanent erosion control effort	Cannot always be placed when needed because of construction traffic and final grading and dressing Initial cost is high

TABLE 3 (continued)
Typical Erosion Control Practice

Treatment Practice	Advantages	Problems
ROADWAY SURFACE		
CROWNING TO DITCH OR SLOPING TO SINGLE BERM 	Directing the surface water to a prepared or protected ditch minimizes erosion	Requires good construction procedures Can cause local stability problems (sloughing)
COMPACTION	The final lift of each day's work should be well compacted and bladed to drain to ditch or berm section Loose or uncompacted material is more subject to erosion	Requires good construction procedures
AGGREGATE COVER 	Minimizes surface erosion Permits construction traffic during adverse weather May be used as part of permanent base construction	Requires reworking and compaction if exposed for long periods of time Loss of surface aggregates can be anticipated
SEED/MULCH 	Minimizes surface erosion	Must be removed or is lost when construction of pavement is commenced
STONE FILLED GABION WALL 	Permits steeper slope No special backfill required Self draining	High cost Requires special techniques to install properly

TABLE 3 (continued)
Typical Erosion Control Practice

Treatment Practice	Advantages	Problems
CUT SLOPES		
BERM AT TOP OF CUT 	Diverts water from cut Collects water for slope drains/paved ditches May be constructed before grading is started	Access to top of cut Difficult to build on steep natural slope or rock surface Concentrates water and may require channel protection or energy dissipation devices Can cause water to enter ground, resulting in sloughing of the cut slope
DIVERSION DIKE 	Collects and diverts water at a location selected to reduce erosion potential May be incorporated in the permanent project drainage	Access for construction May be continuing maintenance problem if not paved or protected Disturbed material or berm is easily eroded
SLOPE BENCHES 	Slows velocity of surface runoff Collects sediment Provides access to slope for seeding, mulching, and maintenance Collects water for slope drains or may divert water to natural ground	May cause sloughing of slopes if water infiltrates Requires additional construction area Not always possible due to poor material, etc. Requires maintenance to be effective Increases excavation quantities
SLOPE DRAINS (PIPE, PAVED, ETC) 	Prevents erosion on the slope Can be temporary or part of permanent construction Can be constructed or extended as grading progresses	Requires supporting effort to collect water Permanent construction is not always compatible with other project work Usually requires some type of energy dissipation

TABLE 3 (continued)
Typical Erosion Control Practice

Treatment Practice	Advantages	Problems
CUT SLOPES (continued)		
SEEDING/MULCHING 	The end objective is to have a completely grassed slope. Early placement is a step in this direction. The mulch provides temporary erosion protection until grass is rooted. Temporary or permanent seeding may be used. Mulch should be anchored. Larger slopes can be seeded and mulched with smaller equipment if stage techniques are used.	Difficult to schedule high production units for small increments Time of year may be less desirable May require supplemental water Contractor may perform this operation with untrained or unexperienced personnel and inadequate equipment if stage seeding is required
SODDING 	Provides immediate protection Can be used to protect adjacent property from sediment and turbidity	Difficult to place until cut is complete Sod not always available May be expensive
SLOPE PAVEMENT, RIPRAP 	Provides immediate protection for high risk areas and under structures May be cast in place or off site	Expensive Difficult to place on high slopes May be difficult to maintain
TEMPORARY COVER 	Plastics are available in wide rolls and large sheets that may be used to provide temporary protection for cut or fill slopes Easy to place and remove Useful to protect high risk areas from temporary erosion	Provides only temporary protection Original surface usually requires additional treatment when plastic is removed Must be anchored to prevent wind damage

TABLE 3 (continued)
Typical Erosion Control Practice

Treatment Practice	Advantages	Problems
CUT SLOPES (continued)		
SERRATED SLOPE 	Lowers velocity of surface runoff Collects sediment Holds moisture Minimizes amount of sediment reaching roadside ditches	May cause minor sloughing if water infiltrates Construction compliance
FABRIC MATS 	Effective for moderate to high embankment when crown vetch plantings are used Has lower cost features over other methods	Requires anchoring time to promote plant growth. May require periodic maintenance
BORROW AREAS		
SELECTIVE GRADING AND SHAPING	Water can be directed to minimize off-site damage Flatter slopes enable mulch to be cut into soil	May not be most economical work method for contractor
STRIPPING AND REPLACING OF TOPSOIL	Provides better seed bed Conventional equipment can be used to stockpile and spread topsoil	May restrict volume of material that can be obtained for a site Topsoil stockpiles must be located to minimize sediment damage Cost of rehandling material
DIKES, BERMS DIVERSION DITCHES SETTLING BASINS SEDIMENT TRAPS SEEDING & MULCH	See other practices	See other practices

TABLE 4
Limiting Flow Velocities to Minimize Erosion

	PERMISSIBLE VELOCITY			
		With Channel Vegetation		
Soil Type	Bare Channel	6" to 10" in height	11" to 24" in height	Over 30" in height
Sand, Silt, Sandy loam, Silty loam	1.5	2.0 to 3.0	2.5 to 3.5	3.0 to 4.0
Silty clay loam, Silty clay	2.0	3.0 to 4.0	3.5 to 4.5	4.0 to 5.0
Clay	2.5	3.0 to 5.0	3.0 to 5.5	3.0 to 6.0

5. SEDIMENT CONTROL. Typical sediment control practices are included in Table 3.

a. Traps. Traps are small and temporary, usually created by excavating and/or diking to a maximum height of five feet. Traps should be cleaned periodically.

b. Ponds.

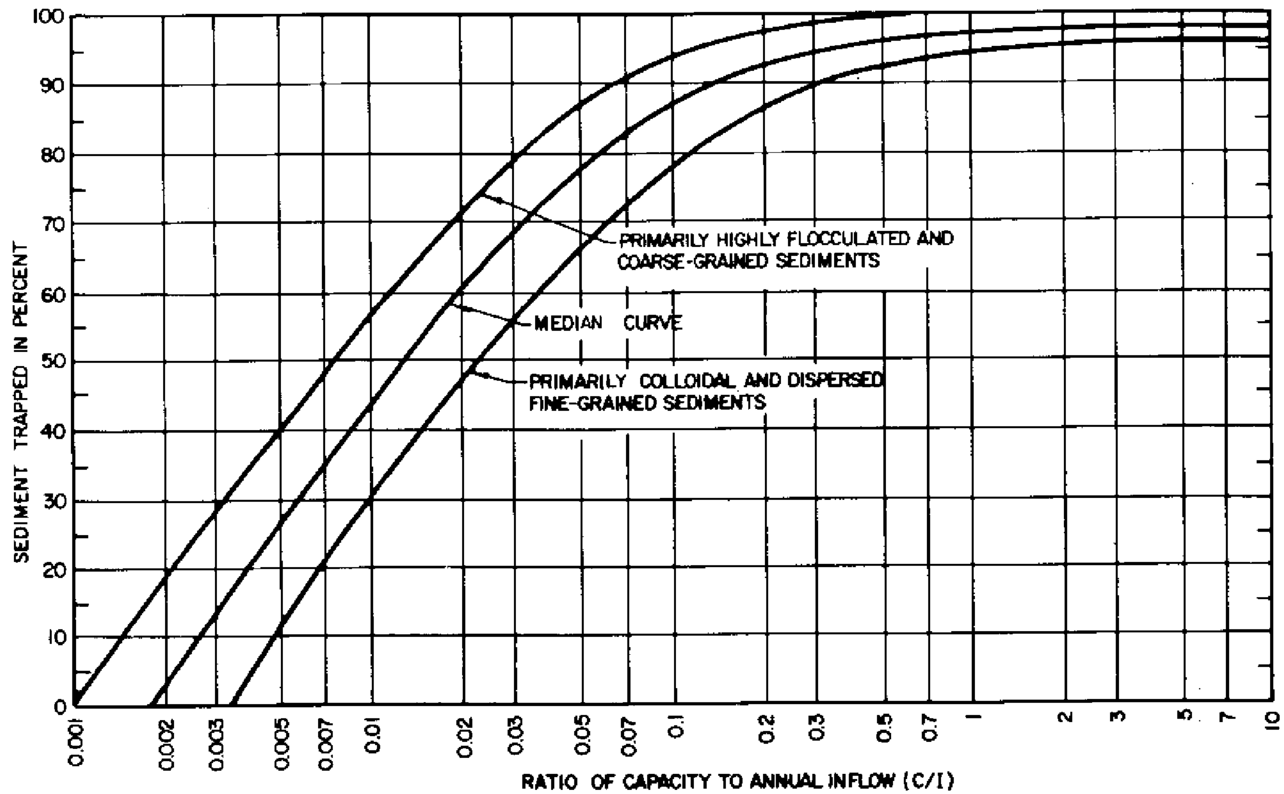
(1) Size the outlet structure to accept the design storm.

(2) Size the pond length, width and depth to remove the desired percentage of sediment. See Figure 13 (modified after Reference 15, Trap Efficiency of Reservoirs, by Brune). For design criteria see Reference 16, Reservoir Sedimentation, by Gottschalk.

(3) If pond is permanent, compute volume of anticipated average annual sedimentation by the Universal Soil Loss Equation. Multiply by the number of years between pond cleaning and by a factor of safety. This equals minimum required volume below water level. Dimensions of the pond can then be calculated based on the available area. The design depth of the pond should be approximately three to five feet greater than the calculated depth of sediment at the time of clearing.

6. RIPRAP PROTECTION. Frequently coarse rock is placed on embankments where erodible soils must be protected from fast currents and wave action. When coarse rock is used, currents and waves may wash soil out from under the rock and lead to undermining and failure. Soil loss under rock slopes can be prevented by the use of filter fabrics or by the placement of a filter layer of intermediate sized material between the soil and rock. In some cases soil loss can be prevented by the use of well-graded rock containing suitable fines which work to the bottom during placement. For further guidance see Reference 17, Tentative Design Procedure for Rip Rap Lined Channels, by the Highway Research Board.

For determining rock sizes and filter requirements use Figure 14 (Reference 18, Design of Small Dams, by the Bureau of Reclamation).



NOTES:

1. FIND ANNUAL SEDIMENT FLOW (FROM UNIVERSAL SOIL LOSS EQUATION.)
2. GENERALLY ASSUME SEDIMENT OF VARIABLE COMPOSITION, USE MEDIAN CURVE.
3. COMPUTE ANNUAL VOLUME OF RUN OFF. (SEE TEXT.)
4. SELECT % OF SEDIMENT TO BE TRAPPED, ENTER CHART FOR C/I.
5. SOLVE FOR C, REQUIRED CAPACITY OF POND. (C AND I MUST HAVE SAME UNITS.)

FIGURE 13
Capacity of Sediment Control Ponds

Example Calculation:

Annual soil loss in watershed = 0.9 acre-feet/year (from Universal Soil Loss Equation or other method, i.e. design charts)

Desired pond efficiency = 70% or 0.63 acre-feet of sediment trapped each year.

Annual volume of runoff from watershed draining into proposed pond = 400 acre-feet/yr.

For 70% efficiency using median curve C/I = 0.032
Required pond capacity C = 0.032 x 400 = 12.8 acre-feet.

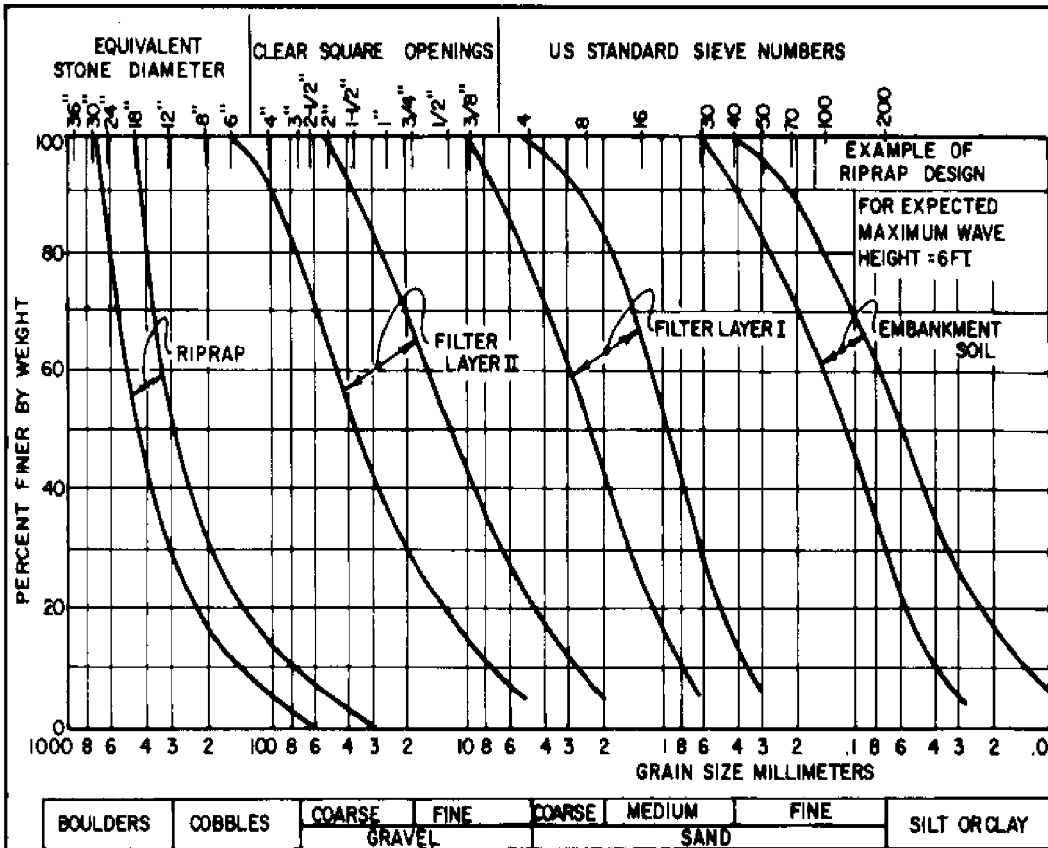
Assuming average depth of pond of 6 ft, required pond area about 2.1 acres. Pond should be cleaned when capacity reduced 50%.

(Note: Trap efficiency decreases as volume of pond decreases; this has not been considered in the example.)

Volume available for sediment = 50% x 12.8 = 6.4 acre-feet.

Years between cleaning = $\frac{6.4}{0.63}$ [approximately] 10 years.

FIGURE 13 (continued)
Capacity of Sediment Control Ponds



GENERAL REQUIREMENTS:

1. FOR EMBANKMENT SLOPES BETWEEN 1:2 AND 1:4 DUMPED RIPRAP SHALL MEET THE FOLLOWING CRITERIA:

MAXIMUM WAVE HEIGHT FT.	AVERAGE ROCK SIZE D_{50}, IN.	MAXIMUM ROCK SIZE POUNDS	LAYER THICKNESS IN.
0 TO 1	8	100	12
1 TO 2	10	200	15
2 TO 4	12	500	18
4 TO 6	15	1500	24
6 TO 8	18	2500	30
8 TO 10	24	4000	36

2. RIPRAP SHALL BE WELL GRADED FROM A MAXIMUM SIZE AT LEAST 1.5 TIMES AVERAGE ROCK SIZE, TO 1 IN. SPALLS SUITABLE TO FILL VOIDS BETWEEN ROCKS.
3. RIPRAP BLANKET SHALL EXTEND TO AT LEAST 8 FT. BELOW LOWEST LOW WATER.
4. UNDER THE MOST EXTREME ICING AND TEMPERATURE CHANGES, ROCK SHOULD MEET SOUNDNESS AND DENSITY REQUIREMENTS FOR CONCRETE AGGREGATE. OTHERWISE, ANY UNWEATHERED ROCK WITH $G > 2.60$, OTHER THAN ARGILLACEOUS TYPES, ARE SUITABLE.
5. FILTER SHALL BE PROVIDED BETWEEN RIPRAP AND EMBANKMENT SOILS TO MEET THE FOLLOWING CRITERIA:

MAXIMUM WAVE HEIGHT, FT.	FILTER D_{85} SIZE AT LEAST:
0 TO 4	1 TO 1-1/2 IN.
4 TO 10	1-1/2 TO 2 IN.

NO FILTER IS NEEDED IF EMBANKMENT MEETS THE ABOVE REQUIREMENTS FOR D_{85} SIZE.

FIGURE 14
Design Criteria for Riprap and Filter on Earth Embankments

FILTER MAY NOT BE REQUIRED IF EMBANKMENT CONSISTS OF CH OR CL WITH LL) 30, RESISTANT TO SURFACE EROSION. IF A FILTER IS USED IN THIS CASE IT ORDINARILY MEETS FILTER CRITERIA AGAINST RIPRAP ONLY.

IF EMBANKMENT CONSISTS OF NONPLASTIC SOILS WHERE SEEPAGE WILL MOVE FROM EMBANKMENT AT LOW WATER, 2 FILTER LAYERS MAY BE REQUIRED WHICH SHALL MEET FILTER CRITERIA AGAINST BOTH EMBANKMENT AND RIPRAP. (EXAMPLE IS SHOWN ABOVE).

6. MINIMUM THICKNESS OF SINGLE LAYER FILTERS ARE AS FOLLOWS:

DOUBLE FILTER LAYERS SHOULD BE AT LEAST 6 IN. THICK.

MAXIMUM WAVE HEIGHT, FT.	FILTER THICKNESS, IN.
0 TO 4	6
4 TO 8	9
8 TO 12	12

FIGURE 14 (continued)
Design Criteria for Riprap and Filter on Earth Embankments

REFERENCES

1. Lee, E. W., Security from Under Seepage of Masonary Dams on Earth Foundations, Transactions, ASCE, Volume 100, Paper 1919, 1935.

2. Marsland, A., Model Experiments to Study the Influence of Seepage on the Stability of a Sheeted Excavation in Sand, Geotechnique, 1952-1953.

3. Calhoun, C. C., Jr., Compton, J. R., Strohm, W. E. Jr., Performance of Plastic Filter Cloths as a Replacement for Granular Materials, Highway Research Record Number 373, Highway Research Board, 1971.

4. Koerner, R. M. and Welsh, J. P., Construction and Geotechnical Engineering Using Synthetic Fabrics, John Wiley & Sons, Inc., 1980.

5. Barber, E. W., Subsurface Drainage of Highways, Highway Research Board Bulletin 209, Highway Research Board, Washington, D.C.

6. Cedergen, H. R., Seepage Requirements of Filters and Pervious Bases, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 86, No. SM5, 1960.

7. Kirkham, D., Seepage Into Ditches From a Plane Water Table Overlying a Gravel Substratum, Journal of Geophysical Research, American Geophysical Union, Washington, D.C., April, 1960.

8. Kirkham, D., Seepage Into Ditches in the Case of a Plane Water Table and an Impervious Substratum, Transactions, American Geophysical Union, Washington, D.C., June, 1950.

9. Avery, S. B., Analysis of Groundwater Lowering Adjacent to Open Water, Proceedings, ASCE, Vol 77, 1951.

10. Corps of Engineers, Soil Mechanics Design, Seepage Control, Engineering Manual, Civil Works Construction, Chapter I, Part CXIX, Department of the Army.

11. Cold Regions Research and Engineering Laboratory, Wastewater Stabilization Pond Linings, Special Report 28, Department of the Army, November, 1978.

12. Soil Conservation Service, U. S. Department of Agriculture, Urban Hydrology for Small Watersheds, Technical Release No. 55, Engineering Division, 1975.

13. Highway Research Board, Erosion Control on Highway Construction, National Cooperative Highway Research Program, Synthesis of Highway Practice 18, 1973.

14. Soil Conservation Service, U. S. Department of Agriculture, Minimizing Erosion in Urbanizing Areas, Madison, WI, 1972.

15. Brune, G. M., Trap Efficiency of Reservoirs, Transactions, American Geophysical Union, Volume 34, No. 3, June, 1953.

16. Gottschalk, L. C., Reservoir Sedimentation, Handbook of Applied Hydrology, Chow, Ed., Section 17-I, McGraw-Hill Book Company, 1964.

17. Highway Research Board, Tentative Design Procedure for Rip-Rap - Lined Channels, National Cooperative Highway Research Program Report 108, Washington, DC, 1970.

18. Bureau of Reclamation, Design of Small Dams, U.S. Department of the Interior, U. S. Government Printing Office, 1973.

19. Naval Facilities Engineering Command, Design Manuals (DM) and Publications (P).

DM-5.03	Drainage Systems
DM-21.06	Airfield Pavement Design for Frost Conditions and Subsurface Drainage
P-418	Dewatering and Groundwater Control

Copies of design manuals and publications may be obtained from the U. S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, Pennsylvania 19120.

PAGE 308 INTENTIONALLY BLANK

CHAPTER 7. SLOPE STABILITY AND PROTECTION

Section 1. INTRODUCTION

1. SCOPE. This chapter presents methods of analyzing stability of natural slopes and safety of embankments. Diagrams are included for stability analysis, and procedures for slope stabilization are discussed.

2. APPLICATIONS. Overstressing of a slope, or reduction in shear strength of the soil may cause rapid or progressive displacements. The stability of slopes may be evaluated by comparison of the forces resisting failure with those tending to cause rupture along the assumed slip surface. The ratio of these forces is the factor of safety.

3. RELATED CRITERIA. Excavations, Earth Pressures, Special Problems - See DM-7.2, Chapters 1, 2 and 3 and DM-7.3, Chapter 3.

4. REFERENCE. For detailed treatment on subject see Reference 1, Landslide Analyses and Control, by the Transportation Research Board.

Section 2. TYPES OF FAILURES

1. MODES OF SLOPE FAILURE. Principal modes of failure in soil or rock are (i) rotation on a curved slip surface approximated by a circular arc, (ii) translation on a planar surface whose length is large compared to depth below ground, and (iii) displacement of a wedge-shaped mass along one or more planes of weakness. Other modes of failure include toppling of rockslopes, falls, block slides, lateral spreading, earth and mud flow in clayey and silty soils, and debris flows in coarse-grained soils. Tables 1 and 2 show examples of potential slope failure problems in both natural and man-made slopes.

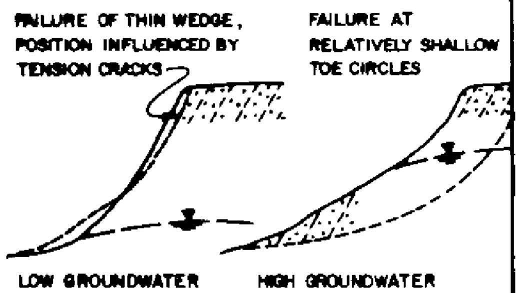
2. CAUSES OF SLOPE FAILURE. Slope failures occur when the rupturing force exceeds resisting force.

a. Natural Slopes. Imbalance of forces may be caused by one or more of the following factors:

(1) A change in slope profile that adds driving weight at the top or decreases resisting force at the base. Examples include steepening of the slope or undercutting of the toe.

(2) An increase of groundwater pressure, resulting in a decrease of frictional resistance in cohesionless soil or swell in cohesive material. Groundwater pressures may increase through the saturation of a slope from rainfall or snowmelt, seepage from an artificial source, or rise of the water table.

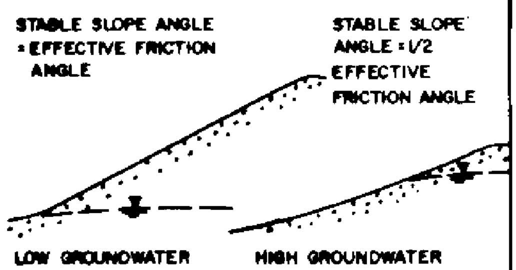
TABLE 1
Analysis of Stability of Natural Slopes



(1) SLOPE IN COARSE-GRAINED SOIL WITH SOME COHESION

WITH LOW GROUNDWATER, FAILURE OCCURS ON SHALLOW, STRAIGHT, OR SLIGHTLY CURVED SURFACE. PRESENCE OF A TENSION CRACK AT THE TOP OF THE SLOPE INFLUENCES FAILURE LOCATION. WITH HIGH GROUNDWATER, FAILURE OCCURS ON THE RELATIVELY SHALLOW TOE CIRCLE WHOSE POSITION IS DETERMINED PRIMARILY BY GROUND ELEVATION.

ANALYZE WITH EFFECTIVE STRESSES USING STRENGTHS c' AND ϕ' FROM CD TESTS. PORE PRESSURE IS GOVERNED BY SEEPAGE CONDITION. INTERNAL PORE PRESSURES AND EXTERNAL WATER PRESSURES MUST BE INCLUDED.

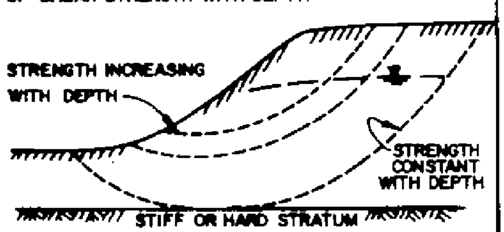


(2) SLOPE IN COARSE-GRAINED, COHESIONLESS SOIL

STABILITY DEPENDS PRIMARILY ON GROUNDWATER CONDITIONS. WITH LOW GROUNDWATER, FAILURES OCCUR AS SURFACE SLOUGHING UNTIL SLOPE ANGLE FLATTENS TO FRICTION ANGLE. WITH HIGH GROUNDWATER, STABLE SLOPE IS APPROXIMATELY 1/2 FRICTION ANGLE.

ANALYZE WITH EFFECTIVE STRESSES USING STRENGTH ϕ'. SLIGHT COHESION APPEARING IN TEST ENVELOPE IS IGNORED. SPECIAL CONSIDERATION MUST BE GIVEN TO POSSIBLE FLOW SLIDES IN LOOSE, SATURATED FINE SANDS.

LOCATION OF FAILURE DEPENDS ON VARIATION OF SHEAR STRENGTH WITH DEPTH



(3) SLOPE IN NORMALLY CONSOLIDATED OR SLIGHTLY PRECONSOLIDATED CLAY

FAILURE OCCURS ON CIRCULAR ARCS WHOSE POSITION IS GOVERNED BY THEORY, SEE FIG. 3. POSITION OF GROUNDWATER TABLE DOES NOT INFLUENCE STABILITY UNLESS ITS FLUCTUATION CHANGES STRENGTH OF THE CLAY OR ACTS IN TENSION CRACKS.

ANALYZE WITH TOTAL STRESSES, ZONING CROSS SECTION FOR DIFFERENT VALUES OF SHEAR STRENGTHS. DETERMINE SHEAR STRENGTH FROM UNCONFINED COMPRESSION TEST, UNCONSOLIDATED UNDRAINED TRIAXIAL TEST OR VANE SHEAR.

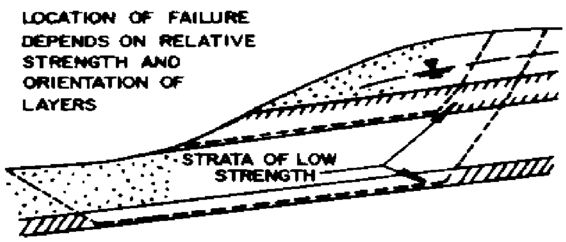
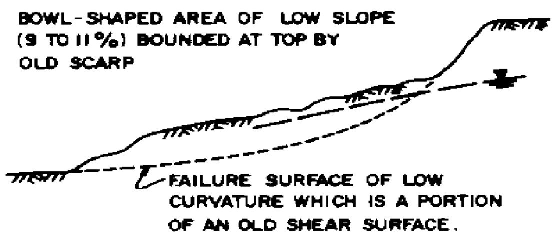
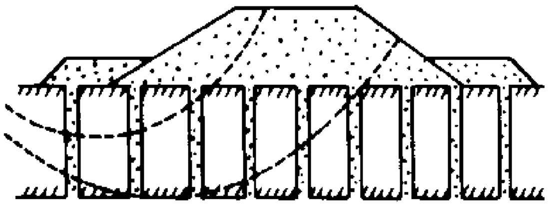
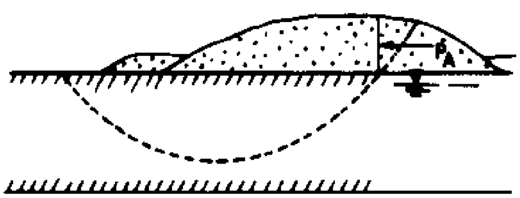
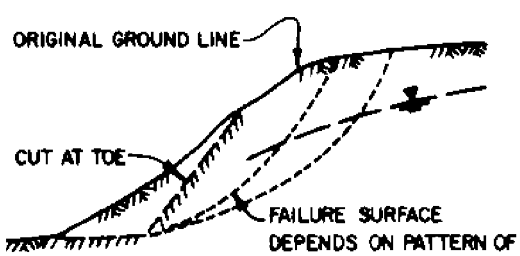
 (4) SLOPE IN STRATIFIED SOIL PROFILE	LOCATION OF FAILURE PLANE IS CONTROLLED BY RELATIVE STRENGTH AND ORIENTATION OF STRATA. FAILURE SURFACE IS COMBINATION OF ACTIVE AND PASSIVE WEDGES WITH CENTRAL SLIDING BLOCK CHOSEN TO CONFORM TO STRATIFICATION. ANALYZE WITH EFFECTIVE STRESS USING c' AND ϕ' FOR FINE-GRAINED STRATA AND ϕ' FOR COHESIONLESS MATERIAL.
 (5) DEPTH CREEP MOVEMENTS IN OLD SLIDE MASS	STRENGTH OF OLD SLIDE MASS DECREASES WITH MAGNITUDE OF MOVEMENT THAT HAS OCCURRED PREVIOUSLY. MOST DANGEROUS SITUATION IS IN STIFF, OVER-CONSOLIDATED CLAY WHICH IS SOFTENED, FRACTURED, OR SLICKENSIDED IN THE FAILURE ZONE.

TABLE 2

Analysis of Stability of Cut and Fill Slopes, Conditions Varying with Time

LOCATION OF FAILURE DEPENDS ON GEOMETRY AND STRENGTH OF CROSS SECTION.  (1) FAILURE OF FILL ON SOFT COHESIVE FOUNDATION WITH SAND DRAINS	USUALLY MINIMUM STABILITY OCCURS DURING PLACING OF FILL. IF RATE OF CONSTRUCTION IS CONTROLLED, ALLOW FOR GAIN IN STRENGTH WITH CONSOLIDATION FROM DRAINAGE. ANALYZE WITH EFFECTIVE STRESS USING c' AND ϕ' FROM CU TEST WITH PORE PRESSURE MEASUREMENT. APPLY ESTIMATED PORE PRESSURES OR PIEZOMETRIC PRESSURES. ANALYZE WITH TOTAL STRESS FOR RAPID CONSTRUCTION WITHOUT OBSERVATION OF PORE PRESSURES, USE SHEAR STRENGTH FROM UNCONFINED COMPRESSION OR UNCONSOLIDATED UNDRAINED TRIAXIAL.
FAILURE SURFACE MAY BE ROTATION ON CIRCULAR ARC OR TRANSLATION WITH ACTIVE AND PASSIVE WEDGES.  (2) FAILURE OF STIFF COMPACTED FILL ON SOFT COHESIVE FOUNDATION	USUALLY, MINIMUM STABILITY OBTAINED AT END OF CONSTRUCTION. FAILURE MAY BE IN THE FORM OF ROTATION OR TRANSLATION, AND BOTH SHOULD BE CONSIDERED. FOR RAPID CONSTRUCTION IGNORE CONSOLIDATION FROM DRAINAGE AND UTILIZE SHEAR STRENGTHS DETERMINED FROM U OR UU TESTS OR VANE SHEAR IN TOTAL STRESS ANALYSIS. IF FAILURE STRAIN OF FILL AND FOUNDATION MATERIALS DIFFER GREATLY, SAFETY FACTOR SHOULD EXCEED ONE, IGNORING SHEAR STRENGTH OF FILL. ANALYZE LONG-TERM STABILITY USING c AND ϕ FROM CU TESTS WITH EFFECTIVE STRESS ANALYSIS, APPLYING PORE PRESSURES OF GROUNDWATER ONLY.
 (3) FAILURE FOLLOWING CUT IN STIFF FISSURED CLAY	RELEASE OF HORIZONTAL STRESSES BY EXCAVATION CAUSES EXPANSION OF CLAY AND OPENING OF FISSURES, RESULTING IN LOSS OF COHESIVE STRENGTH. ANALYZE FOR SHORT TERM STABILITY USING c' AND ϕ' WITH TOTAL STRESS ANALYSIS. ANALYZE FOR LONG TERM STABILITY WITH c'_r AND ϕ'_m BASED ON RESIDUAL STRENGTH MEASURED IN CONSOLIDATED DRAINED TESTS.

(3) Progressive decrease in shear strength of the soil or rock mass caused by weathering, leaching, mineralogical changes, opening and softening of fissures, or continuing gradual shear strain (creep).

(4) Vibrations induced by earthquakes, blasting, or pile-driving. Induced dynamic forces cause densification of loose sand, silt, or loess below the groundwater table or collapse of sensitive clays, causing increased pore pressures. Cyclic stresses induced by earthquakes may cause liquefaction of loose, uniform, saturated sand layers (see DM-7.3, Chapter 1).

b. Embankment (Fill) Slopes. Failure of fill slopes may be caused by one or more of the following factors:

(1) Overstressing of the foundation soil. This may occur in cohesive soils, during or immediately after embankment construction. Usually, the short-term stability of embankments on soft cohesive soils is more critical than the long-term stability, because the foundation soil will gain strength as the pore water pressure dissipates. It may, however, be necessary to check the stability for a number of pore pressure conditions. Usually, the critical failure surface is tangent to the firm layers below the soft subsoils.

(2) Drawdown and Piping. In earth dams, rapid drawdown of the reservoir causes increased effective weight of the embankment soil thus reducing stability. Another potential cause of failure in embankment slopes is subsurface erosion or piping (see Chapter 6 for guidance on prevention of piping).

(3) Dynamic Forces. Vibrations may be induced by earthquakes, blasting, pile driving, etc.

c. Excavation (Cut) Slopes. Failure may result from one or more of the factors described in (a). An additional factor that should be considered for cuts in stiff clays is the release of horizontal stresses during excavation which may cause the formation of fissures. If water enters the fissures, the strength of the clay will decrease progressively. Therefore, the long-term stability of slopes excavated in cohesive soils is normally more critical than the short-term stability. When excavations are open over a long period and water is accessible, there is potential for swelling and loss of strength with time.

3. EFFECT OF SOIL OR ROCK TYPE.

a. Failure Surface. In homogeneous cohesive soils, the critical failure surface usually is deep whereas shallow surface sloughing and sliding is more typical in homogeneous cohesionless soils. In nonhomogeneous soil foundations the shape and location of the failure depends on the strength and stratification of the various soil types.

b. Rock. Slope failures are common in stratified sedimentary rocks, in weathered shales, and in rocks containing platy minerals such as talc, mica, and the serpentine minerals. Failure planes in rock occur along zones of weakness or discontinuities (fissures, joints, faults) and bedding planes (strata). The orientation and strength of the discontinuities are the most important factors influencing the stability of rock slopes. Discontinuities can develop or strength can change as a result of the following environmental factors:

(1) Chemical weathering.

(2) Freezing and thawing of water/ice in joints.

(3) Tectonic movements.

(4) Increase of water pressures within discontinuities.

(5) Alternate wetting and drying (especially expansive shales).

(6) Increase of tensile stresses due to differential erosion.

Further guidance pertinent to rock slopes can be found in DM-7.2, Chapter 1.

Section 3. METHODS OF ANALYSIS

1. TYPES OF ANALYSIS. For slopes in relatively homogeneous soil, the failure surface is approximated by a circular arc, along which the resisting and rupturing forces can be analyzed. Various techniques of slope stability analysis may be classified into three broad categories.

a. Limit Equilibrium Method. Most limit equilibrium methods used in geotechnical practice assume the validity of Coulomb's failure criterion along an assumed failure surface. A free body of the slope is considered to be acted upon by known or assumed forces. Shear stresses induced on the assumed failure surface by the body and external forces are compared with the available shear strength of the material. This method does not account for the load deformation characteristics of the materials in question. Most of the methods of stability analysis currently in use fall in this category.

The method of slices, which is a rotational failure analysis, is most commonly used in limit equilibrium solutions. The minimum factor of safety is computed by trying several circles. The difference between various approaches stems from (a) the assumptions that make the problem determinate, and (b) the equilibrium conditions that are satisfied. The soil mass within the assumed slip surface is divided into several slices, and the forces acting on each slice are considered. The effect of an earthquake may be considered by applying appropriate horizontal force on the slices. Figure 1 (Reference 2, Soil Mechanics, by Lambe and Whitman) illustrates this method of analysis applied to a slope of homogeneous sandy soil subjected to the forces of water seeping laterally toward a drain at the toe.

b. Limit Analysis. This method considers yield criteria and the stress-strain relationship. It is based on lower bound and upper bound theorems for bodies of elastic - perfectly plastic materials. See Reference 3, Stability of Earth Slopes, by Fang, for further guidance.

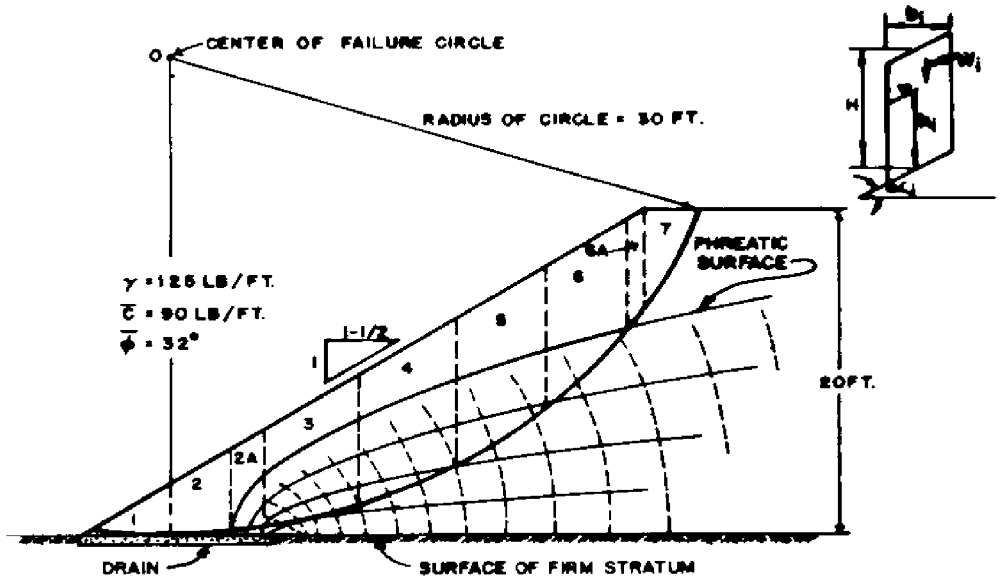
Considering the equilibrium of forces in the vertical direction but neglecting the shearing forces between slices the factor of safety for moment equilibrium becomes (neglecting earthquake forces):

$$F_m = \frac{\sum_{i=1}^{i=N} \left[\bar{c} b_i + (W_i - u_i b_i) \tan \bar{\phi}\right] / M_{\alpha i}}{\sum_{i=1}^{i=N} W_i \sin \alpha_i}$$

$$\text{WHERE} \quad M_{\alpha i} = \cos \alpha_i \left(1 + \frac{\tan \alpha_i \tan \bar{\phi}}{F_m}\right)$$

The above equation is solved by successive approximations. Value of $M_{\alpha i}$ is obtained from Figure 1 (continued) Graph for Determination of M_α for an assumed value of F_m.

Example:



Find F_m for the trial slip circle shown.

Properties

$$\bar{c} = 90 \text{ psf}, \quad \bar{\phi} = 32°, \quad \gamma = 125 \text{ PCF}$$

Slope 1-1/2 horizontal to 1 vertical.

Flow conditions as shown.

FIGURE 1
Method of Slices - Simplified Bishop Method (Circular Slip Surface)

Procedure (numbers in parenthesis corresponds to column in example):

1. Divide cross section into vertical slices,(1).
2. Calculate weight of each slice (W_i) using total unit weights, where b_i is the width of the slice and H is the average height of the slice, (2),(3), (4).
3. Calculate $W_i \sin \alpha_i$ for each slice, where α_i is the angle between the tangent of the failure surface and the horizontal, (5)(6).
4. Multiply the cohesive strength ($\bar{c}$) times the width of each slice (b_i), (7).
5. Multiply the average pore water pressure [(u_i) = (h_i)(.0624 KSF)] along the failure surface of each slice, times the width of each slice, (8).
6. Calculate $(W_i - u_i b_i)\tan\bar{\phi}$ for each slice, (9).
7. Add $\bar{c}b_i$ plus $(W_i - u_i b_i)\tan\bar{\phi}$ for each slice, (10).
8. Select two factors of safety (F_m), and find $M\alpha_i$ for each slice using graph below (11).
9. Divide $\bar{c}b_i + (W_i - u_i b_i)\tan\bar{\phi}$ by $M\alpha_i$ for each slice and sum resultants, (12).
10. Divide $\sum_{i=1}^{i=n} \frac{\bar{c}b_i + (W_i - u_i b_i)\ \mathrm{TAN}\bar{\phi}}{M\alpha_i}$ by $\sum_{i=1}^{i=n} W_i \mathrm{SIN}\alpha_i$ to obtain calculated F_m

    Compare to F_m's assumed in Step 8. Reiterate Steps 8, 9, and 10 until assumed F_m of Step 8 equals calculated F_m of Step 10.
11. Repeat above analysis varying center location and radius of failure circle to establish least factor of safety.

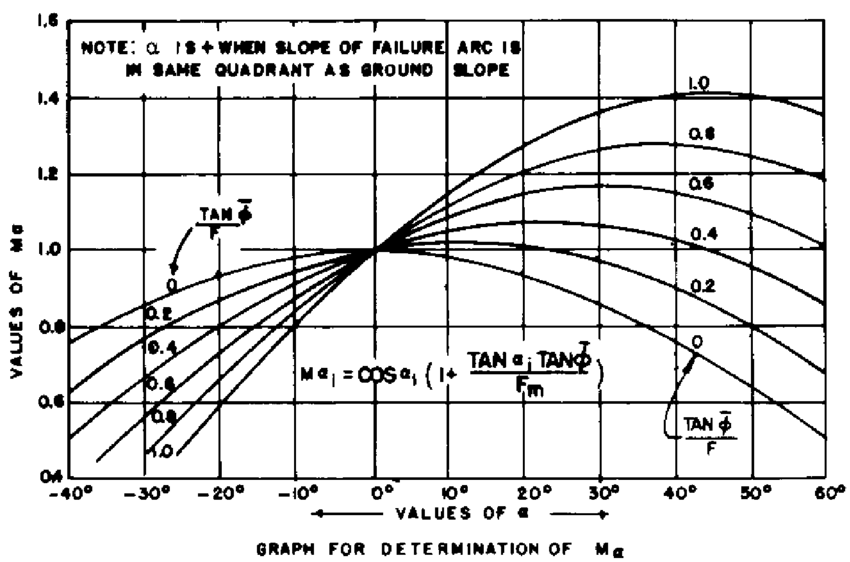


FIGURE 1 (continued)
Method of Slices - Simplified Bishop Method (Circular Slip Surface)

Slice (1)	b_i (FT) (2)	H (FT) (3)	W_i (KIPS) (4)	$\sin\alpha_i$ (5)	$W_i \sin\alpha_i$ (KIPS) (6)	$\bar{c}b_i$ (KIPS) (7)	$u_i b_i$ (KIPS) (8)	$(W_i - u_i b_i)$ $\tan\bar{\phi}$ (KIPS) (9)	(7+9) (KIPS) (10)	$M\alpha_i$ (11) F_m = 1.25	F_m = 1.35	(10) ÷ (11) (12) F_m = 1.25	F_m = 1.35
1	4.5	1.6	0.9	0.03	0	0.40	0	0.55	0.95	0.97	0.97	1.00	1.00
2	3.2	4.2	1.7	0.05	0.1	0.29	0	1.05	1.35	1.02	1.02	1.30	1.30
2A	1.8	5.8	1.3	0.14	0.2	0.16	0.05	0.80	0.95	1.06	1.05	0.90	0.90
3	5.0	7.4	4.6	0.25	1.2	0.45	1.05	2.25	2.70	1.09	1.08	2.50	2.50
4	5.0	9.0	5.6	0.42	2.3	0.45	1.45	2.55	3.00	1.12	1.10	2.70	2.75
5	5.0	9.3	5.8	0.58	3.4	0.45	1.25	2.70	3.15	1.10	1.08	2.85	2.90
6	4.4	8.4	4.6	0.74	3.4	0.40	0.50	2.65	3.05	1.05	1.02	2.90	2.95
6A	0.6	6.7	0.5	0.82	0.4	0.05	0	0.30	0.35	0.98	0.95	0.35	0.40
7	3.2	3.8	1.5	0.87	1.3	0.29	0	0.95	1.25	0.93	0.92	1.30	1.35
					12.3							15.80	16.05

For assumed F_m = 1.25, calculated, $Fm = \frac{15.8}{12.3} = 1.29$

F_m = 1.35, calculated, $Fm = \frac{16.05}{12.3} = 1.31$

A trial assuming F = 1.3 would yield Fm = 1.3

FIGURE 1 (continued)
Method of Slices - Simplified Bishop Method (Circular Slip Surface)

c. Finite Element Method. This method is extensively used in more complex problems of slope stability and where earthquake and vibrations are part of total loading system. This procedure accounts for deformation and is useful where significantly different material properties are encountered.

2. FAILURE CHARACTERISTICS. Table 1 shows some situations that may arise in natural slopes. Table 2 shows situations applicable to man-made slopes. Strength parameters, flow conditions, pore water pressure, failure modes, etc. should be selected as described in Section 4.

3. SLOPE STABILITY CHARTS.

a. Rotational Failure in Cohesive Soils ([phi] = 0)

(1) For slopes in cohesive soils having approximately constant strength with depth use Figure 2 (Reference 4, Stability Analysis of Slopes with Dimensionless Parameters, by Janbu) to determine the factor of safety.

(2) For slope in cohesive soil with more than one soil layer, determine centers of potentially critical circles from Figure 3 (Reference 4). Use the appropriate shear strength of sections of the arc in each stratum. Use the following guide for positioning the circle.

(a) If the lower soil layer is weaker, a circle tangent to the base of the weaker layer will be critical.

(b) If the lower soil layer is stronger, two circles, one tangent to the base of the upper weaker layer and the other tangent to the base of the lower stronger layer, should be investigated.

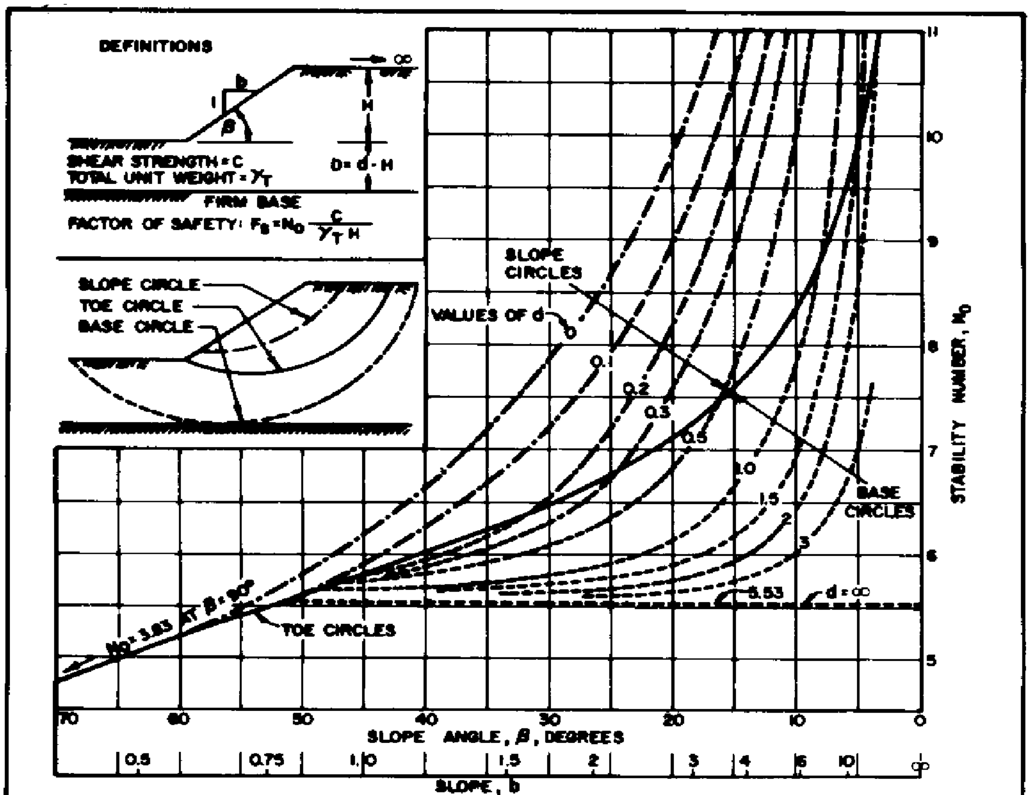
(3) With surcharge, tension cracks, or submergence of slope, apply corrections of Figure 4 to determine safety factor.

(4) Embankments on Soft Clay. See Figure 5 (Reference 5, The Design of Embankments on Soft Clays, by Jakobsen) for approximate analysis of embankment with stabilizing berms on foundations of constant strength. Determine the probable form of failure from relationship of berm and embankment widths and foundation thickness in top left panel of Figure 5.

4. TRANSLATIONAL FAILURE ANALYSIS. In stratified soils, the failure surface may be controlled by a relatively thin and weak layer. Analyze the stability of the potentially translating mass as shown in Figure 6 by comparing the destabilizing forces of the active pressure wedge with the stabilizing force of the passive wedge at the toe plus the shear strength along the base of the central soil mass. See Figure 7 for an example of translational failure analysis in soil and Figure 8 for an example of translational failure in rock.

Jointed rocks involve multiple planes of weakness. This type of problem cannot be analyzed by two-dimensional cross-sections. See Reference 6, The Practical and Realistic Solution of Rock Slope Stability, by Von Thun.

5. REQUIRED SAFETY FACTORS. The following values should be provided for reasonable assurance of stability:



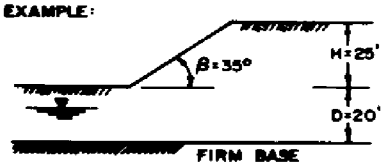
ASSUMED CONDITIONS:

1. NO OPEN WATER OUTSIDE OF SLOPE.
2. NO SURCHARGE OR TENSION CRACKS.
3. SOIL IS HOMOGENEOUS TO DEPTH D.
4. SHEAR STRENGTH IS DERIVED FROM COHESION ONLY AND IS CONSTANT WITH DEPTH.
5. FAILURE TAKES PLACE AS ROTATION ON CIRCULAR ARC.

NOTES:

1. FOR LOCATION OF CENTER OF CRITICAL CIRCLE SEE FIG.3.
2. FOR REDUCTION FACTORS FOR SUBMERGENCE, SURCHARGE OR TENSION CRACKS, SEE FIG.4.

EXAMPLE:



γ_T = 115 PCF C = 600 PSF $\phi = 0$

d = D/H = 20/25 = 0.80, $\beta = 35°$

$F_S = N_O \frac{C}{\gamma_T H} = 5.8 \frac{(600)}{(115)(25)} = \underline{1.21}$

FAILURE SURFACE IS BASE CIRCLE.

FIGURE 2
Stability Analysis for Slopes in Cohesive Soils, Undrained Conditions, i.e., Assumed $\phi = 0$

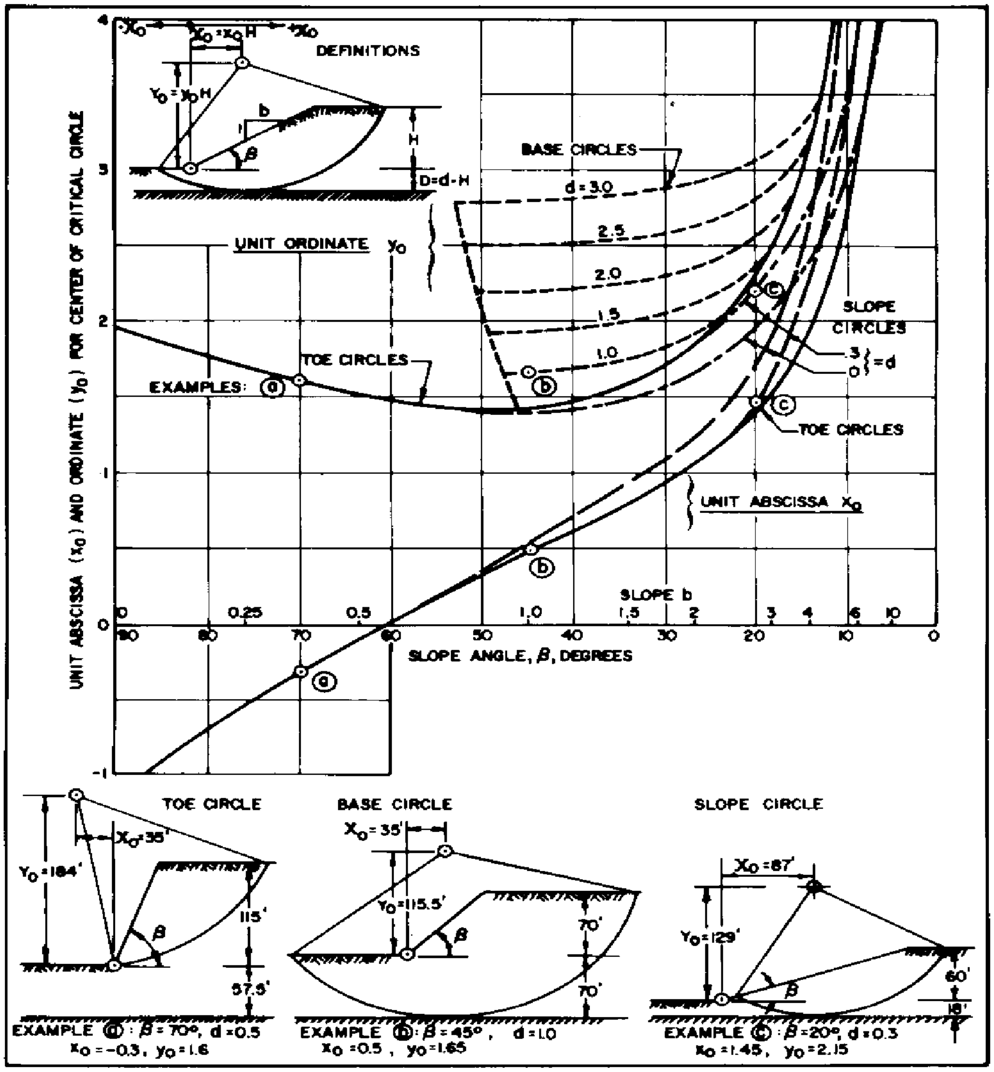


FIGURE 3
Center of Critical Circle, Slope in Cohesive Soil

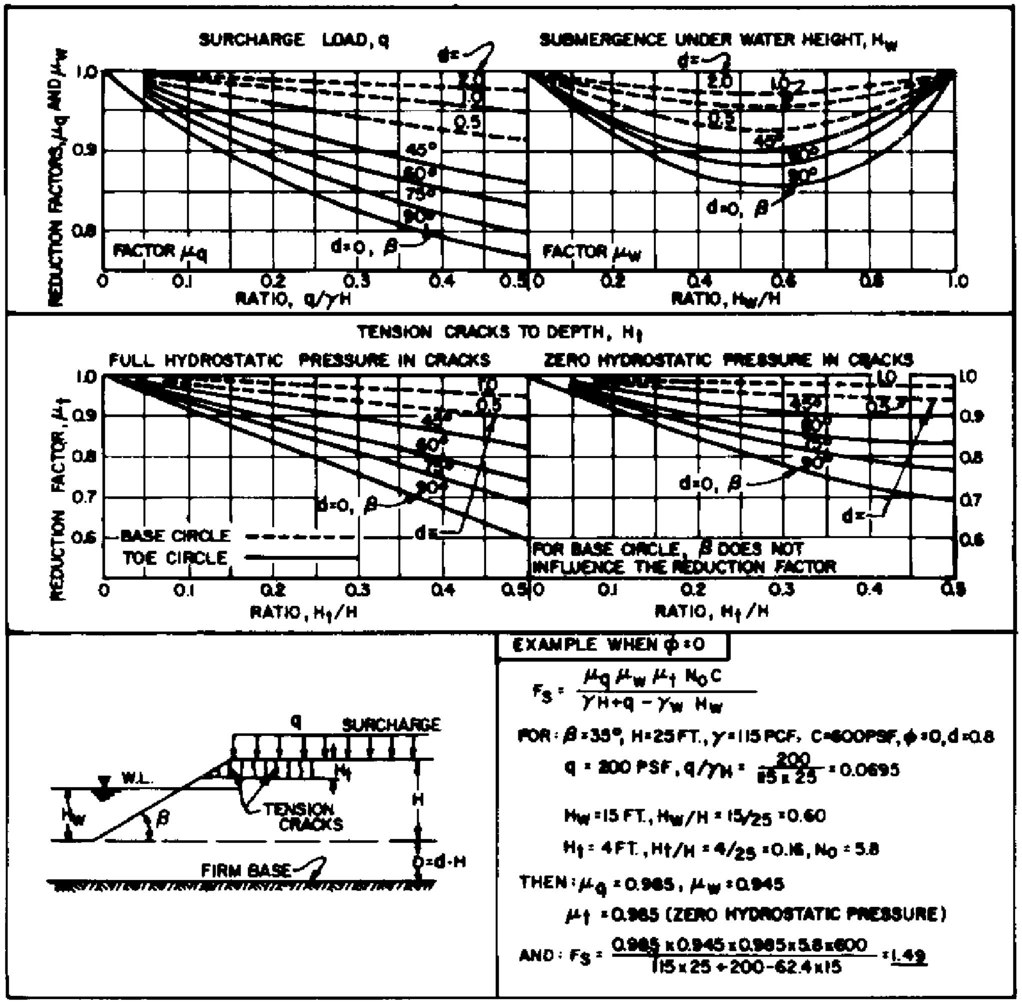


FIGURE 4
Influence of Surcharge, Submergence, and Tension Cracks on Stability

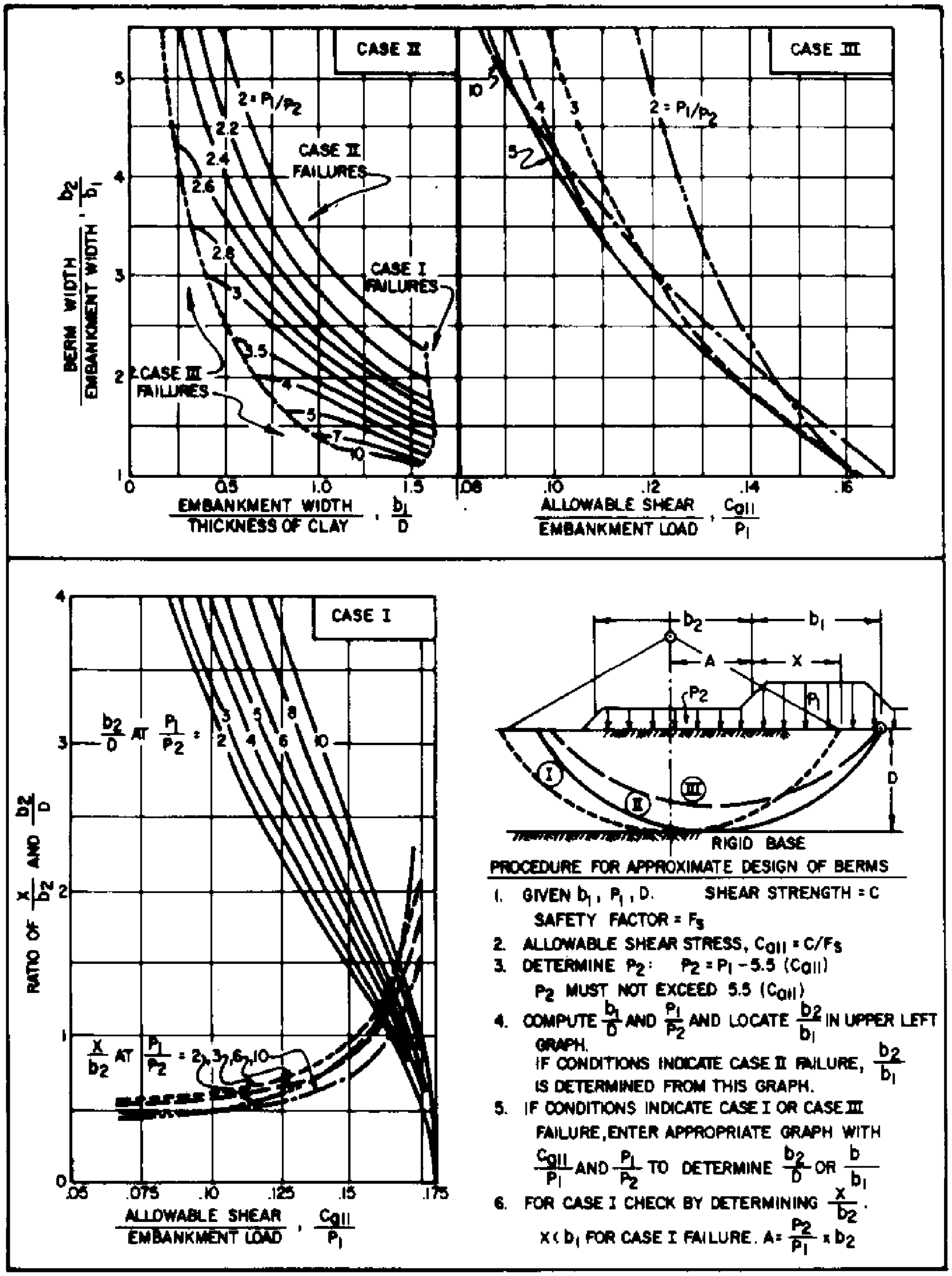


FIGURE 5
Design of Berms for Embankments on Soft Clays

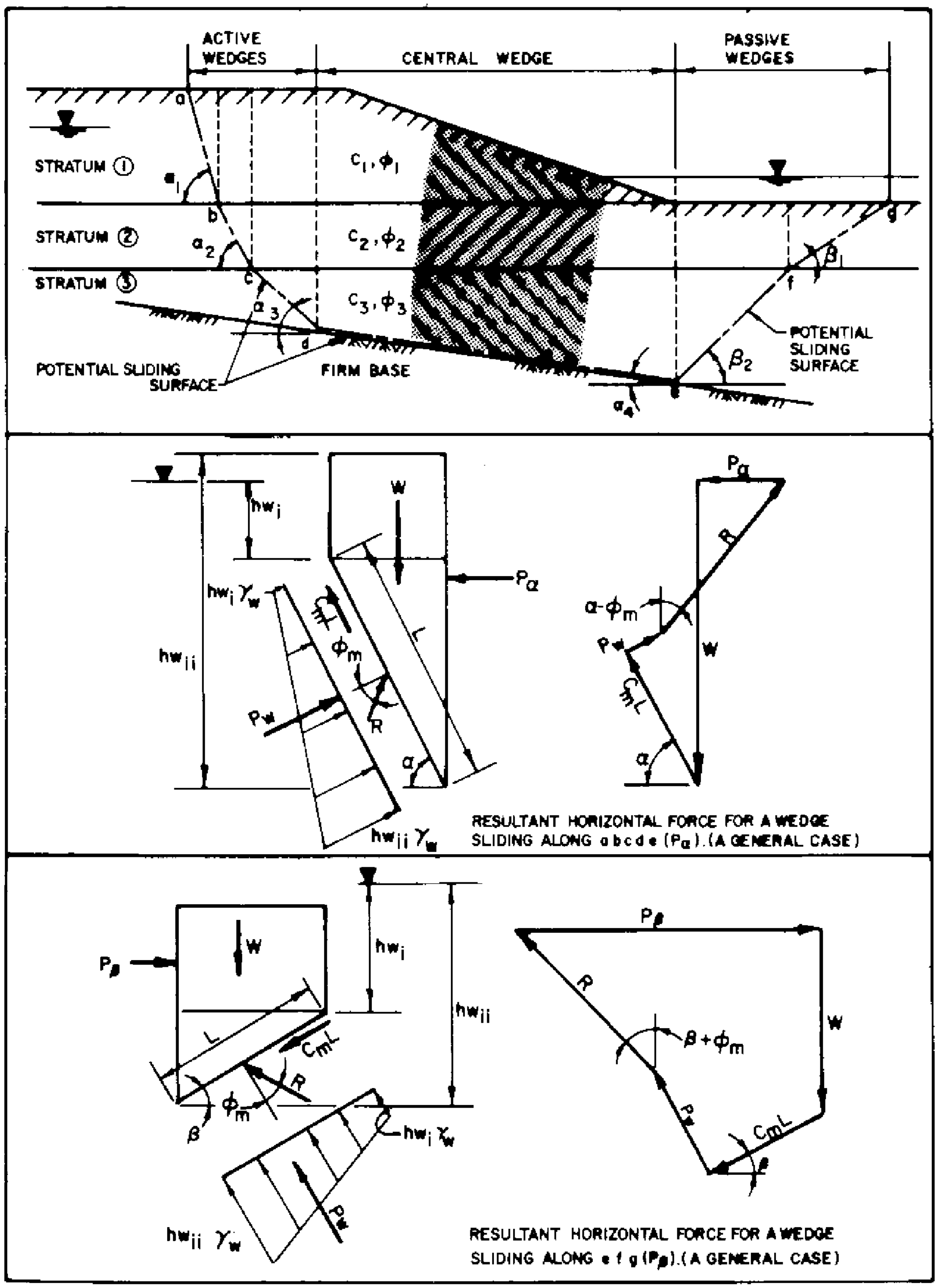


FIGURE 6
Stability Analysis of Translational Failure

DEFINITION OF TERMS

P_α = RESULTANT HORIZONTAL FORCE FOR AN ACTIVE OR CENTRAL WEDGE ALONG POTENTIAL SLIDING SURFACE a b c d e.

P_β = RESULTANT HORIZONTAL FORCE FOR A PASSIVE WEDGE ALONG POTENTIAL SLIDING SURFACE e f g.

W = TOTAL WEIGHT OF SOIL AND WATER IN WEDGE ABOVE POTENTIAL SLIDING SURFACE.

R = RESULT OF NORMAL AND TANGENTIAL FORCES ON POTENTIAL SLIDING SURFACE CONSIDERING FRICTION ANGLE OF MATERIAL.

P_w = RESULTANT FORCE DUE TO PORE WATER PRESSURE ON POTENTIAL SLIDING SURFACE CALCULATED AS:

$$P_w = \left[\frac{hw_i + hw_{ii}}{2}\right](L)(\gamma_w)$$

ϕ = FRICTION ANGLE OF LAYER ALONG POTENTIAL SLIDING SURFACE.

C = COHESION OF LAYER ALONG POTENTIAL SLIDING SURFACE.

L = LENGTH OF POTENTIAL SLIDING SURFACE ACROSS WEDGE.

h_w = DEPTH BELOW PHREATIC SURFACE AT BOUNDARY OF WEDGE.

γ_w = UNIT WEIGHT OF WATER.

PROCEDURES

1.) EXCEPT FOR CENTRAL WEDGE WHERE α IS DICTATED BY STRATIGRAPHY USE $\alpha = 45° + \frac{\phi}{2}$, $\beta = 45° - \frac{\phi}{2}$ FOR ESTIMATING FAILURE SURFACE.

2.) SOLVE FOR P_α AND P_β FOR EACH WEDGE IN TERMS OF THE SAFETY FACTOR (F_s) USING THE EQUATIONS SHOWN BELOW. THE SAFETY FACTOR IS APPLIED TO SOIL STRENGTH VALUES (TAN ϕ AND C).

MOBILIZED STRENGTH PARAMETERS ARE THEREFORE CONSIDERED AS $\phi_m = \text{TAN}^{-1}\left(\frac{\text{TAN}\,\phi}{F_s}\right)$ AND $C_m = \frac{C}{F_s}$.

$$P_\alpha = \left[W - C_m L \text{ SIN } \alpha - P_w \text{ COS } \alpha\right] \text{TAN}\left[\alpha - \phi_m\right] - \left[C_m L \text{ COS } \alpha - P_w \text{ SIN } \alpha\right]$$

$$P_\beta = \left[W + C_m L \text{ SIN } \beta - P_w \text{ COS } \beta\right]\left[\text{TAN } (\beta + \phi_m)\right] + \left[C_m L \text{ COS } \beta + P_w \text{ SIN } \beta\right]$$

IN WHICH THE FOLLOWING EXPANSIONS ARE TO BE USED:

$$\text{TAN } (\alpha - \phi_m) = \frac{\text{TAN } \alpha - \frac{\text{TAN } \phi}{F_s}}{1 + \text{TAN } \alpha \frac{\text{TAN } \phi}{F_s}} \qquad \text{TAN } (\beta + \phi_m) = \frac{\text{TAN } \beta + \frac{\text{TAN } \phi}{F_s}}{1 - \text{TAN } \beta \frac{\text{TAN } \phi}{F_s}}$$

3.) FOR EQUILIBRIUM $\Sigma P_\alpha = \Sigma P_\beta$. SUM P_α AND P_β FORCES IN TERMS OF F_s, SELECT TRIAL F_s, CALCULATE ΣP_α AND ΣP_β. IF $\Sigma P_\alpha \neq \Sigma P_\beta$, REPEAT. PLOT P_α AND P_β VS. F_s WITH SUFFICIENT TRIALS TO ESTABLISH THE POINT OF INTERSECTION (i.e., $\Sigma P_\alpha = \Sigma P_\beta$), WHICH IS THE CORRECT SAFETY FACTOR.

4.) DEPENDING ON STRATIGRAPHY AND SOIL STRENGTH, THE CENTER WEDGE MAY ACT TO MAINTAIN OR UPSET EQUILIBRIUM.

5.) NOTE THAT FOR $\phi = 0$, ABOVE EQUATIONS REDUCE TO:

$$P_\alpha = W \text{ TAN } \alpha - \frac{C_m L}{\text{COS } \alpha}, \quad P_\beta = W \text{ TAN } \beta + \frac{C_m L}{\text{COS } \beta}$$

6.) THE SAFETY FACTOR FOR SEVERAL POTENTIAL SLIDING SURFACES MAY HAVE TO BE COMPUTED IN ORDER TO FIND THE MINIMUM SAFETY FACTOR FOR THE GIVEN STRATIGRAPHY.

FIGURE 6 (continued)
Stability Analysis of Translational Failure

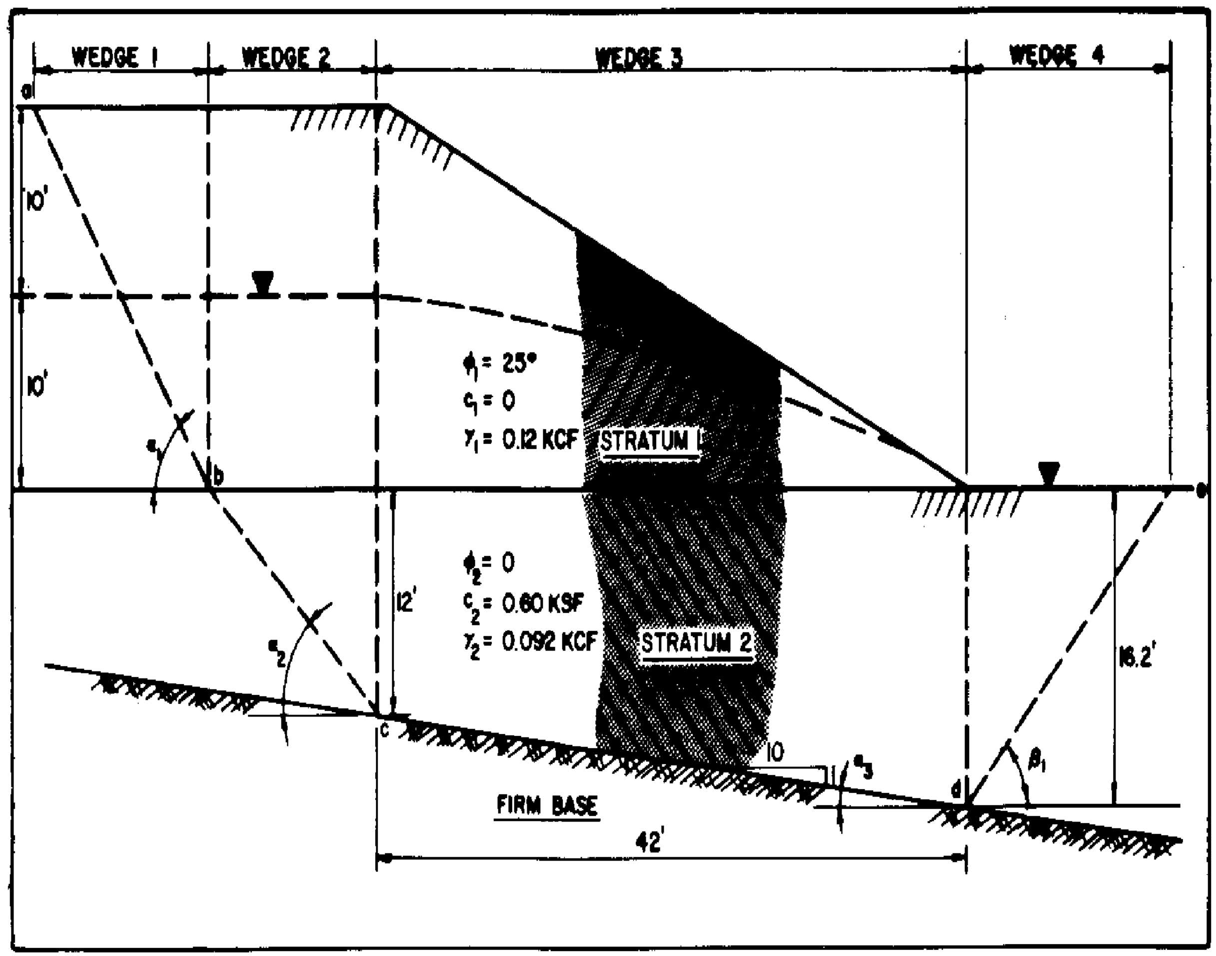


FIGURE 7
Example of Stability Analysis of Translational Failure

FORCES P_a

WEDGE 1: $\phi = 25°, C = 0, \gamma = 0.12$ KCF (SLIDING SURFACE ab)

$\alpha_1 = 45 + \phi_1/2 = 57.5°$

$W = \frac{20}{2} \times 20 \text{ TAN } 32.5° \times 0.12 = 15.29$ KIPS

$P_w = \left(\frac{0+10}{2}\right)(0.062) \times \left(\frac{10}{\text{SIN } 57.5°}\right) = 3.68$ KIPS

$$P_{a1} = (W - P_w \text{ COS } \alpha_1)\frac{\left(\text{TAN } \alpha_1 - \frac{\text{TAN } \phi_1}{F_S}\right)}{\left(1 + \text{TAN } \alpha_1 \frac{\text{TAN } \phi_1}{F_S}\right)} + P_w \text{ SIN } \alpha_1$$

$$= (15.29 - 1.98)\frac{\left(1.57 - \frac{0.47}{F_S}\right)}{\left(1 + \frac{0.73}{F_S}\right)} + 3.10 = \left(\frac{20.90 F_S - 6.26}{F_S + 0.73}\right) + 3.10$$

WEDGE 2: $\phi = 0, C = 0.60$ KSF, $\gamma = 0.092$ KCF (SLIDING SURFACE bc)

$\alpha_2 = 45°$

$W = 12 \times 10 \times 0.12 + 12 \times 10 \times 0.12 + \frac{12}{2} \times 12 \times 0.092 = 35.42$ KIPS

$$P_{a2} = W \text{ TAN } \alpha_2 - \frac{\frac{CL}{F_S}}{\text{COS } \alpha_2} \quad (\text{FOR } \phi = 0)$$

$$= 35.42 - \frac{\left(\frac{0.60}{F_S} \times \frac{12}{.707}\right)}{(.707)} = 35.42 - \frac{14.40}{F_S}$$

WEDGE 3: $\phi = 0, C = 0.60$ KSF, $\gamma = 0.092$ KCF (SLIDING SURFACE cd)

$\alpha_3 = \text{TAN}^{-1}\ 0.1 = 5.7°$

$W = \frac{20}{2} \times 42 \times 0.12 + \frac{12 + 16.2}{2} \times 42 \times 0.092 = 104.88$ KIPS

$$P_{a3} = W \text{ TAN } \alpha_3 - \frac{\frac{CL}{F_S}}{\text{COS } \alpha} \quad (\text{FOR } \phi = 0)$$

$$= \boxed{104.9 \times 0.10} - \left[\frac{\left(\frac{0.60}{F_S} \times \frac{42}{0.99}\right)}{0.99}\right] = 10.49 - \frac{25.71}{F_S}$$

$$\Sigma P_a = \frac{20.90 F_S - 6.26}{F_S + 0.73} + 49.01 - \frac{40.11}{F_S}$$

FORCES P_β

WEDGE 4: $\phi = 0, C = 0.60$ KSF, $\gamma = 0.092$ KCF (SLIDING SURFACE de)

$\beta_1 = 45°$

$W = \frac{16.2}{2} \times 16.2 \times 0.092 = 12.07$ KIPS

$$P_{\beta 1} = W \text{ TAN } \beta + \frac{\frac{CL}{F_S}}{\text{COS } \beta} \quad (\text{FOR } \phi = 0)$$

$$= 12.07 + \left[\frac{\frac{0.80}{F_S} \times \frac{16.20}{0.707}}{0.707}\right] = 12.07 + \frac{19.44}{F_S}$$

$$\Sigma P_\beta = 12.07 + \frac{19.44}{F_S}$$

FIGURE 7 (continued)
Example of Stability Analysis of Translational Failure

SOLVE FOR F_S, FROM $\Sigma P_\alpha = \Sigma P_\beta$

F_s	ΣP_α	ΣP_β
1.0	17.4	31.5
1.1	21.7	29.7
1.2	25.3	28.3
1.3	28.5	27.0

$F_s = 1.27$

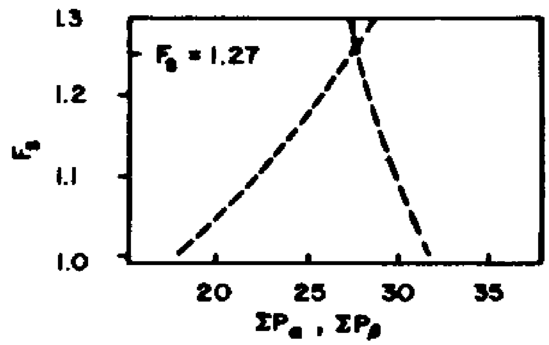


FIGURE 7 (continued)
Example of Stability Analysis of Translational Failure

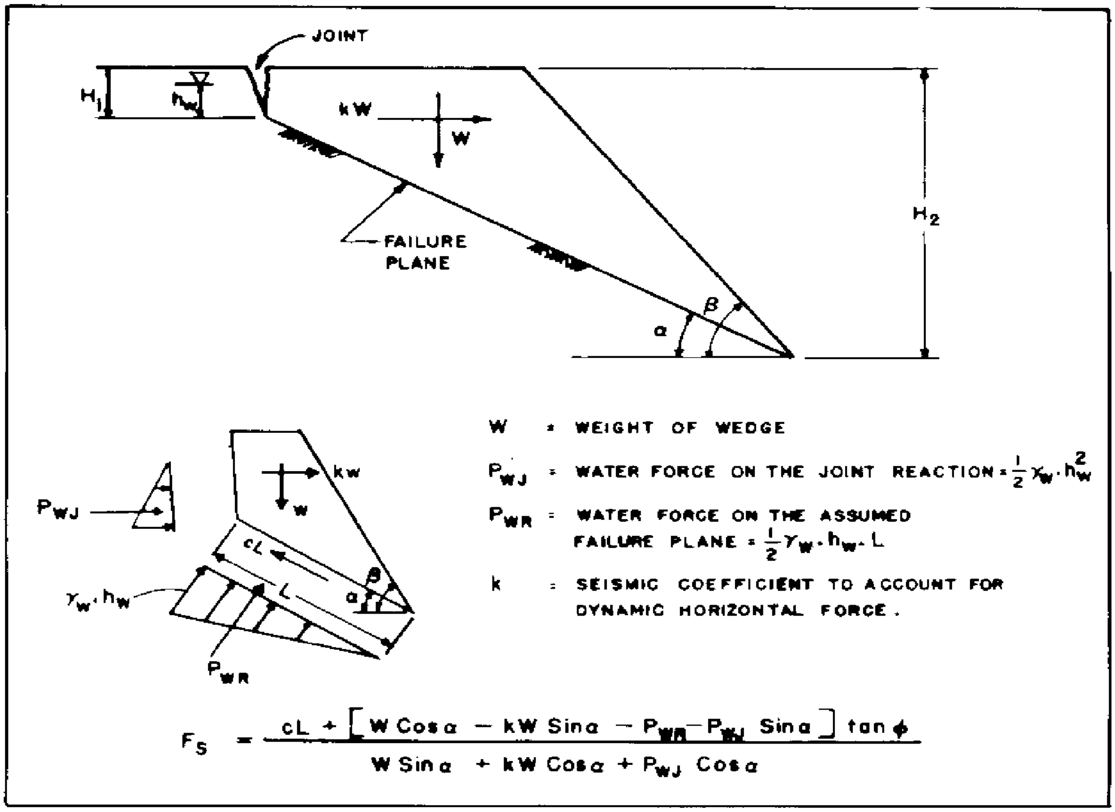


FIGURE 8
Stability of Rock Slope

(1) Safety factor no less than 1.5 for permanent or sustained loading conditions.

(2) For foundations of structures, a safety factor no less than 2.0 is desirable to limit critical movements at foundation edge. See DM-7.2, Chapter 4 for detailed requirements for safety factors in bearing capacity analysis.

(3) For temporary loading conditions or where stability reaches a minimum during construction, safety factors may be reduced to 1.3 or 1.25 if controls are maintained on load application.

(4) For transient loads, such as earthquake, safety factors as low as 1.2 or 1.15 may be tolerated.

6. EARTHQUAKE LOADING. Earthquake effects can be introduced into the analysis by assigning a disturbing force on the sliding mass equal to kW where W is the weight of the sliding mass and k is the seismic coefficient. For the analyses of stability shown in Figure 9a, $k_s W$ is assumed to act parallel to the slope and through the center of mass of the sliding mass. Thus, for a factor of safety of 1.0:

$$Wb + k_s Wh = FR$$

The factor of safety under an earthquake loading then becomes

$$F_{Se} = \frac{FR}{Wb + k_s Wh}$$

To determine the critical value of the seismic efficient (k_{cs}) which will reduce a given factor of safety for a stable static condition (F_{So}) to a factor of safety of 1.0 with an earthquake loading ($F_{Se} = 1.0$), use

$$k_{cs} = \frac{b}{h}(F_{So} - 1) = (F_{So} - 1)\sin\theta$$

If the seismic force is in the horizontal direction and denoting such force as $k_{ch} W$, then $k_{ch} = (F_{So} - 1)\tan\theta$.

For granular, free-draining material with plane sliding surface (Figure 9b): $F_{So} = \tan\phi/\tan\theta$, and $k_{cs} = (F_{So} - 1)\sin\theta$.

Based on several numerical experiments reported in Reference 7, <u>Critical Acceleration Versus Static Factor of Safety in Stability Analysis of Earth Dams and Embankments</u>, by Sarma and Bhave, k_{ch}, may be conservatively represented as $k_{ch} \approx (F_{So} - 1)0.25$.

The downslope movement U may be conservatively predicted based on Reference 8, <u>Effect of Earthquakes on Dams and Embankments</u>, by Newmark as:

$$U = \frac{V^2}{2g\,k_{cs}} \times \frac{A}{k_{cs}}$$

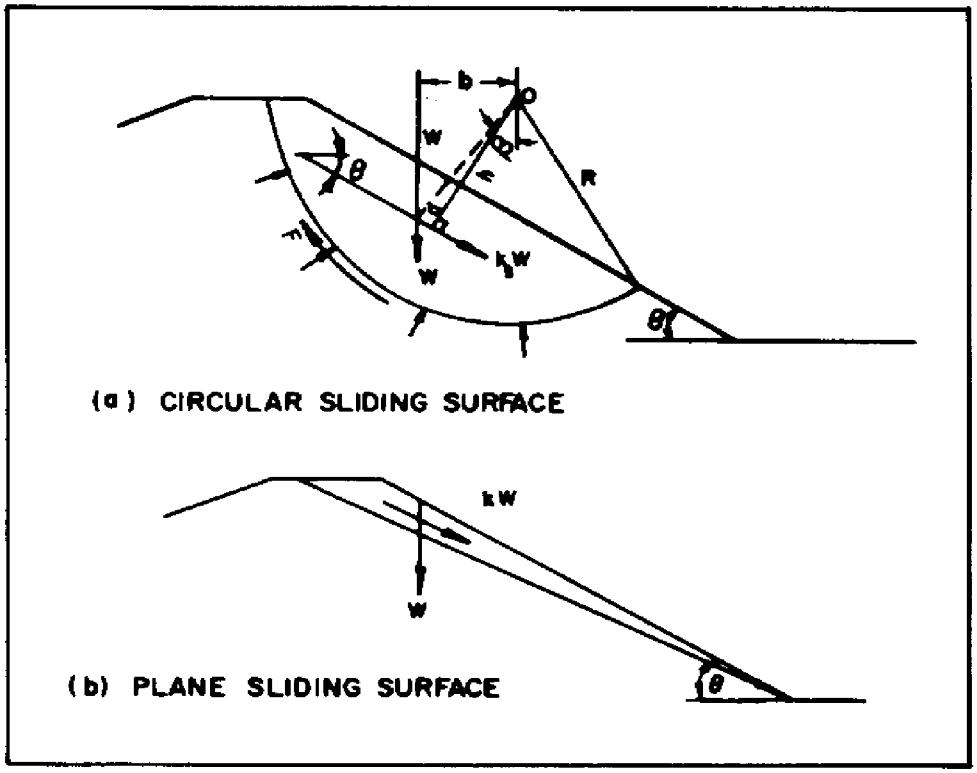


FIGURE 9
Earthquake Loading on Slopes

where A = a peak ground acceleration, g's

g = a acceleration of gravity

V = peak ground velocity

The above equations are based on several simplifying assumptions: (a) failure occurs along well defined slip surface, (b) the sliding mass behaves as a rigid body; (c) soils are not sensitive and would not collapse at small deformation; and (d) there is no reduction in soil strength due to ground shaking.

Section 4. EFFECTS OF SOIL PARAMETERS AND GROUNDWATER ON STABILITY

1. INTRODUCTION. The choice of soil parameters and the methods of analyses are dictated by the types of materials encountered, the anticipated groundwater conditions, the time frame of construction, and climatic conditions. Soil strength parameters are selected either on the basis of total stress, ignoring the effect of the pore water pressure, or on the basis of effective stress where the analysis of the slope requires that the pore water pressures be treated separately.

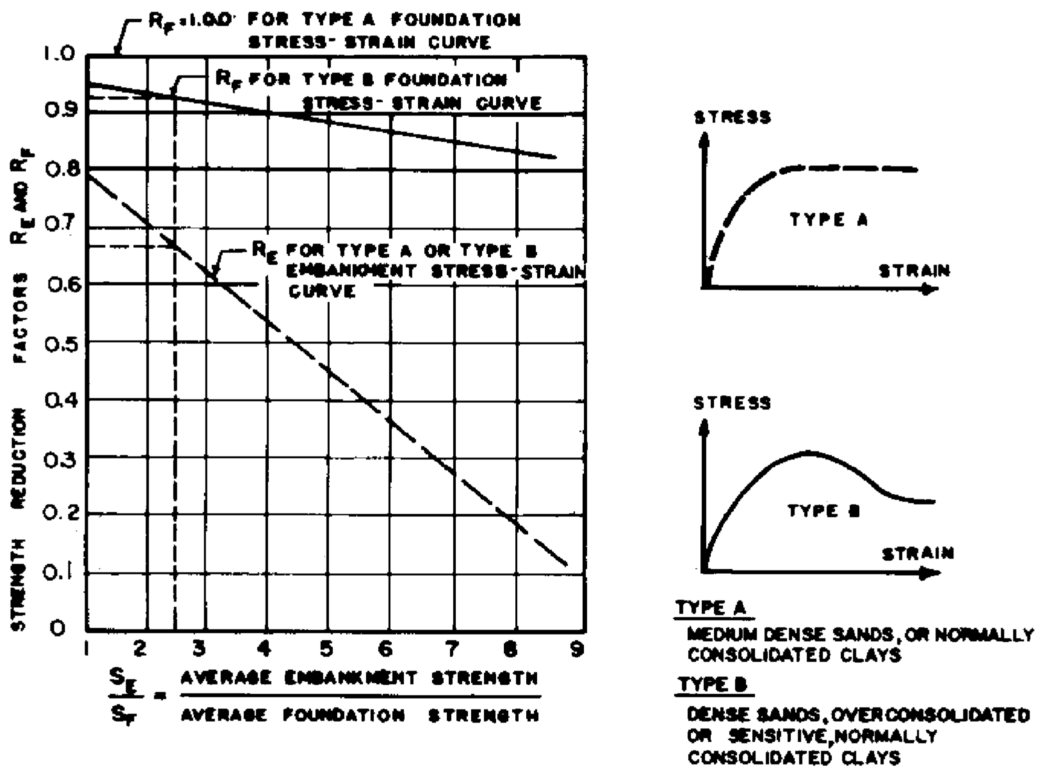
2. TOTAL VS. EFFECTIVE STRESS ANALYSIS. The choice between total stress and effective stress parameters is governed by the drainage conditions which occur within the sliding mass and along its boundaries. Drainage is dependent upon soil permeability, boundary conditions, and time.

a. Total Stress Analysis. Where effective drainage cannot occur during shear, use the undrained shear strength parameters such as vane shear, unconfined compression, and unconsolidated undrained (UU or Q) triaxial compression tests. Field vane shear and cone penetration tests may be used. Assume [phi] = 0. Examples where a total stress analysis are applicable include:

(1) Analysis of cut slopes of normally consolidated or slightly preconsolidated clays. In this case little dissipation of pore water pressure occurs prior to critical stability conditions.

(2) Analysis of embankments on a soft clay stratum. This is a special case as differences in the stress-strain characteristics of the embankment and the foundation may lead to progressive failure. The undrained strength of both the foundation soil and the embankment soil should be reduced in accordance with the strength reduction factors R+E, and R+F, in Figure 10 (Reference 9, An Engineering Manual for Slope Stability Studies, by Duncan and Buchignani).

(3) Rapid drawdown of water level providing insufficient time for drainage. Use the undrained strength corresponding to the overburden condition within the structure prior to drawdown.



EXAMPLES:

FROM LAB: EMBANKMENT SOIL IS TYPE A, UNDRAINED SHEAR STRENGTH = 1000 PSF
FOUNDATION SOIL IS TYPE B, UNDRAINED SHEAR STRENGTH = 400 PSF

THEN $\frac{S_E}{S_F} = \frac{1000}{400} = 2.5$

$R_E = 0.67$

$R_F = 0.93$

USE UNDRAINED SHEAR STRENGTHS $S_E = 670$ AND $S_F = 372$ FOR ANALYSIS

NOTE: UNDRAINED SHEAR STRENGTH IS INDEPENDENT OF TOTAL NORMAL STRESS

FIGURE 10

Correction Factors R_E and R_F to Account for Progressive Failure in Embankments on Soft Clay Foundations

(4) End-of-construction condition for fills built of cohesive soils. Use the undrained strength of samples compacted to field density and at water content representative of the embankment.

b. Effective Stress Analysis. The effective shear strength parameters (c', [phi]') should be used for the following cases:

(1) Long-term stability of clay fills. Use steady state seepage pressures where applicable.

(2) Short-term or end-of-construction condition for fills built of free draining sand and gravel. Friction angle is usually approximated by correlation for this case. See Chapter 1.

(3) Rapid drawdown condition of slopes in pervious, relatively incompressible, coarse-grained soils. Use pore pressures corresponding to new lower water level with steady state flow.

(4) Long-term stability of cuts in saturated clays. Use steady state seepage pressures where applicable.

(5) Cases of partial dissipation of pore pressure in the field. Here, pore water pressures must be measured by piezometers or estimated from consolidation data.

3. EFFECT OF GROUNDWATER AND EXCESS PORE PRESSURE. Subsurface water movement and associated seepage pressures are the most frequent cause of slope instability. See Table 1 for illustrations of the effects of water on slope stability.

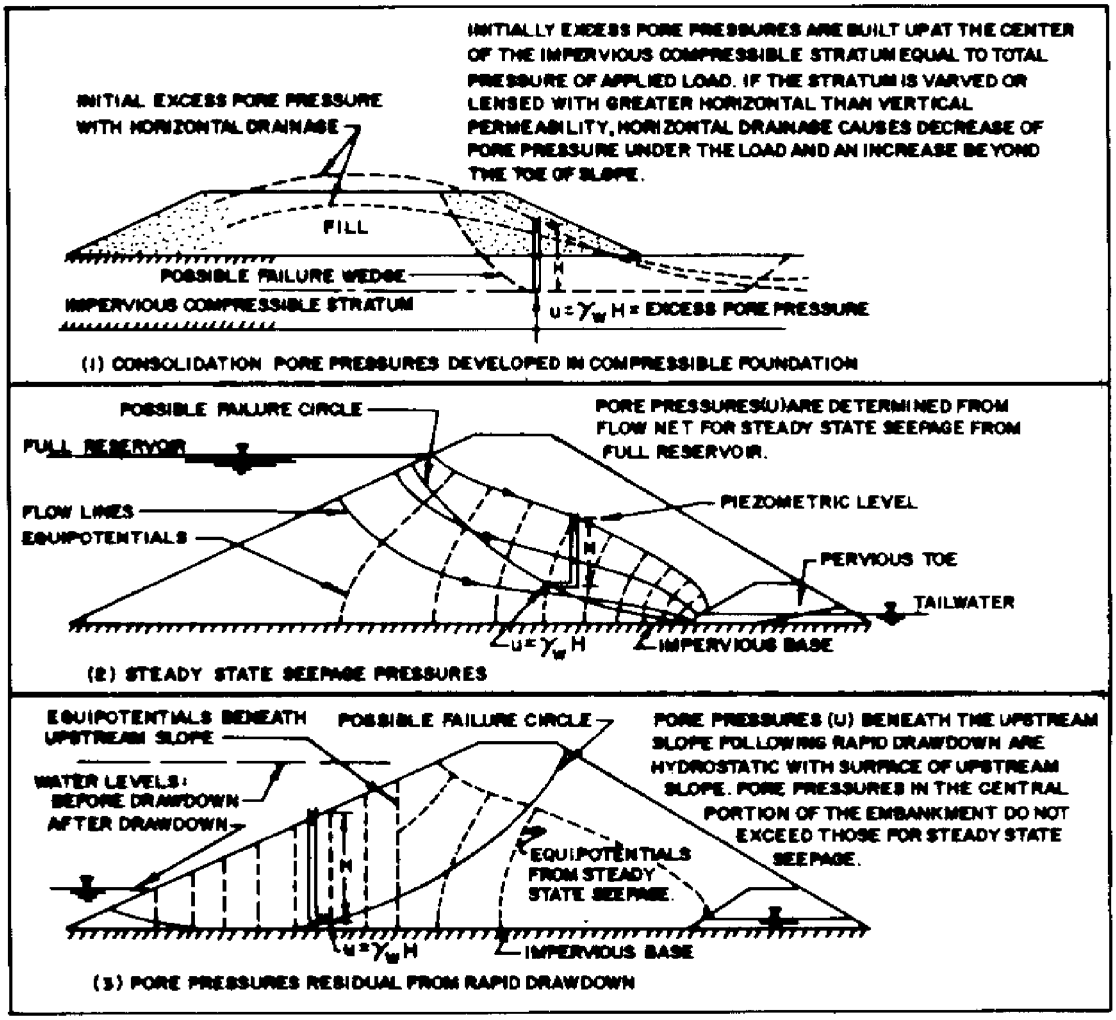
a. Seepage Pressures. Subsurface water seeping toward the face or toe of a slope produces destabilizing forces which can be evaluated by flow net construction. The piezometric heads which occur along the assumed failure surface produce outward forces which must be considered in the stability analysis. See Table 3 and the example of Figure 1.

b. Construction Pore Pressures. When compressible fill materials are used in embankment construction, excess pore pressure may develop and must be considered in the stability analysis. Normally, field piezometric measurements are required to evaluate this condition.

c. Excess Pore Pressures in Embankment Foundations. Where embankments are constructed over compressible soils, the foundation pore pressures must be considered in the stability analysis. See top panel of Table 3.

d. Artesian Pressures. Artesian pressures beneath slopes can have serious effects on the stability. Should such pressures be found to exist, they must be used to determine effective stresses and unit weights, and the slope and foundation stability should be evaluated by effective stress methods.

TABLE 3
Pore Pressure Conditions for Stability Analysis Homogeneous Embankment



4. STABILITY PROBLEMS IN SPECIAL MATERIALS

a. Controlling Factors. See Table 1, DM-7.2, Chapter 1, for primary factors controlling slope stability in some special problem soils.

b. Strength Parameters.

(1) Overconsolidated, Fissured Clays and Clayshales. See Table 2. Cuts in these materials cause opening of fissures and fractures with consequent softening and strength loss.

(a) Analysis of Cut Slopes. For long-term stability of cut slopes use residual strength parameters c'+r, and [phi]'+r, from drained tests. See Chapter 3. The most reliable strength information for fissured clays is frequently obtained by back figuring the strength from local failures.

(b) Old Slide Masses. Movements in old slide masses frequently occur on relatively flat slopes because of gradual creep at depth. Exploration may show the failure mass to be stiff or hard; but a narrow failure plane of low strength with slickensides or fractures may be undetected. In such locations avoid construction which involves regrading or groundwater rise that may upset a delicate equilibrium.

(2) Saturated Granular Soils in Seismic Areas. Ground shaking may result in liquefaction and strength reduction of certain saturated granular soils. Empirical methods are available for estimating the liquefaction potential. See DM-7.3, Chapter 1 for guidance. Methods of stabilization for such soils are discussed in DM-7.3, Chapter 2.

(3) Loess and Other Collapsible soils. Collapse of the structure of these soils can cause a reduction of cohesion and a rise in pore pressure.

Evaluate the saturation effects with unconsolidated undrained tests, saturating samples under low chamber pressure prior to shear. See Chapter 1 for evaluating collapse potential.

(4) Talus. For talus slopes composed of friable material, [phi] may range from 20deg. to 25deg. If consisting of debris derived from slate or shale, [phi] may range from 20deg. to 29deg., limestone about 32deg., gneiss 34deg., granite 35deg. to 40deg. These are crude estimates of friction angles and should be supplemented by analysis of existing talus slopes in the area.

Section 5. SLOPE STABILIZATION

1. METHODS. See Table 4, for a summary of slope stabilization methods. A description of some of these follows:

a. Regrading Profile. Flattening and/or benching the slope, or adding material at the toe, as with the construction of an earth berm, will increase the stability. Analyze by procedures above to determine most effective regrading.

TABLE 4
Methods of Stabilizing Excavation Slopes

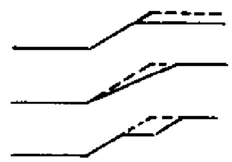
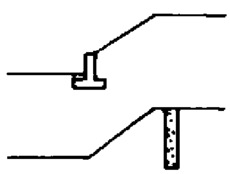
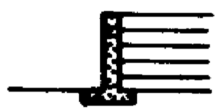
Scheme	Applicable Methods	Comments
1. Changing Geometry EXCAVATION 	1. Reduce slope height by excavation at top of slope. 2. Flatten the slope angle. 3. Excavate a bench in upper part of slope.	1. Area has to be accessible to construction equipment. Disposal site needed for excavated soil. Drainage sometimes incorporated in this method.
2. Earth Berm Fill 	1. Compacted earth or rock berm placed at and beyond the toe. Drainage may be provided behind berm.	1. Sufficient width and thickness of berm required so failure will not occur below or through berm.
3. Retaining Structures RETAINING STRUCTURES 	1. Retaining wall - crib or cantilever type. 2. Drilled, cast-in-place vertical piles, founded well below bottom of slide plane. Generally 18 to 36 inches in diameter and 4- to 8-foot spacing. Larger diameter piles at closer spacing may be required in some cases to mitigate failures of cuts in highly fissured clays.	1. Usually expensive. Cantilever walls might have to be tied back. 2. Spacing should be such that soil can arch between piles. Grade beam can be used to tie piles together. Very large diameter (6 feet ±) piles have been used for deep slides.

TABLE 4 (continued)
Methods of Stabilizing Excavation Slopes

Scheme	Applicable Methods	Comments
	3. Drilled, cast-in-place vertical piles tied back with battered piles or a deadman. Piles founded well below slide plane. Generally, 12 to 30 inches in diameter and at 4- to 8-foot spacing.	3. Space close enough so soil will arch between piles. Piles can be tied together with grade beam.
	4. Earth and rock anchors and rock bolts.	4. Can be used for high slopes, and in very restricted areas. Conservative design should be used, especially for permanent support. Use may be essential for slopes in rocks where joints dip toward excavation, and such joints daylight in the slope.
	5. Reinforced earth.	5. Usually expensive.
4. Other Methods	See Table 7, DM-7.2, Chapter 1	

b. Seepage and Groundwater Control. Surface control of drainage decreases infiltration to potential slide area. Lowering of groundwater increases effective stresses and eliminates softening of fine-grained soils at fissures. Details on seepage and groundwater control are found in Chapter 6.

c. Retaining Structures.

(1) Application. Walls or large diameter piling can be used to stabilize slides of relatively small dimension in the direction of movement or to retain steep toe slopes so that failure will not extend back into a larger mass.

(2) Analysis. Retaining structures are frequently misused where active forces on wall are computed from a failure wedge comprising only a small percentage of the total weight of the sliding mass. Such failures may pass entirely beneath the wall, or the driving forces may be large enough to shear through the retaining structure. Stability analysis should evaluate a possible increase of pressures applied to wall by an active wedge extending far back into failing mass (see Figure 4, DM-7.2, Chapter 3), and possible failure on sliding surface at any level beneath the base of the retaining structure.

(3) Piles or Caissons. To be effective, the piles should extend sufficiently below the failure surface to develop the necessary lateral resistance. Figure 11 shows how the effect of the piles is considered in calculating the factor of safety. The distribution of pressure along the pile can be computed from charts shown in Figure 12. This assumes full mobilization of soil shear strength along the failure surface and should be used only when the safety factor without the piles is less than 1.4. This criteria is based on results of analysis presented in Reference 10, Forces Induced in Piles by Unsymmetrical Surcharges on the Soil Around the Pile, by DeBeer and Wallays.

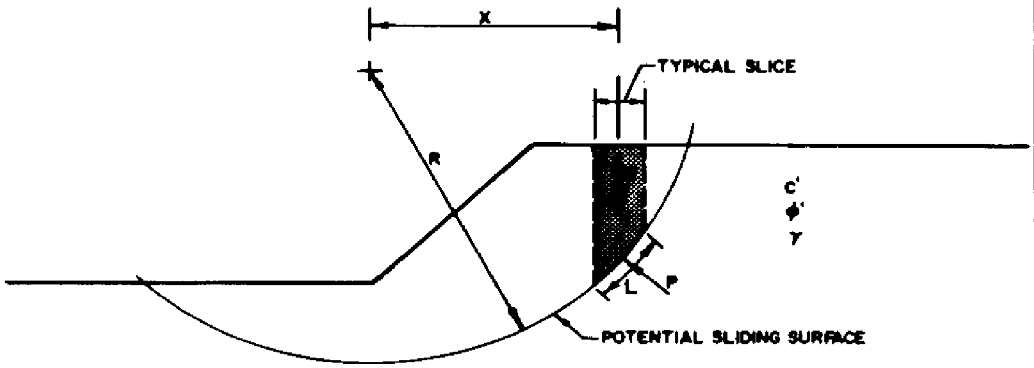
See Figure 13 for example computations. Note the computations shown are for only one of the many possible slip surfaces.

d. Other Methods.

(1) Other potential procedures for stabilizing slopes include grouting, freezing, electro osmosis, vacuum pumping, and diaphragm walls. See Table 7 of DM-7.2, Chapter 1 for further guidance on these methods.

Section 6. SLOPE PROTECTION

1. SLOPE EROSION. Slopes which are susceptible to erosion by wind and rain-fall should be protected. Protection is also required for slopes subjected to wave action as in the upstream slope of a dam, or the river and canal banks along navigational channels. In some cases, provision must be made against burrowing animals.

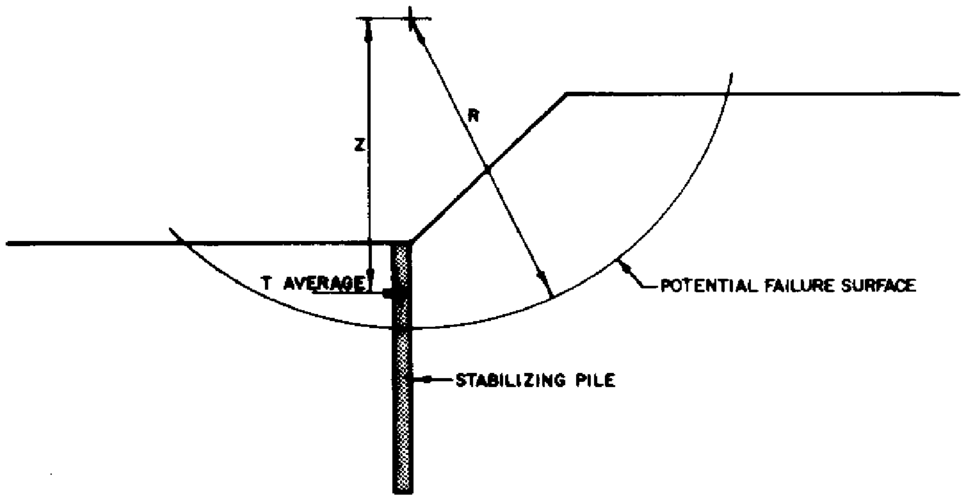


SAFETY FACTOR (F_s) FOR MOMENT EQUILIBRIUM FOR METHOD OF SLICES WITH CIRCULAR POTENTIAL SLIDING SURFACE (WITHOUT SEISMIC LOADS, SURCHARGE LOADS, OR PARTIAL SUBMERGENCE OF SLOPE) IS TYPICALLY DEFINED AS:

$$F_s = \frac{\Sigma\, c' L R + \Sigma (P - uL)\, R \tan \phi'}{\Sigma\, WX}$$

WHERE: c' = EFFECTIVE COHESION.
ϕ' = EFFECTIVE FRICTION ANGLE.
W = TOTAL WEIGHT OF SLICE.
P = TOTAL NORMAL FORCE ON BASE OF SLICE.
L = LENGTH OF POTENTIAL SLIDING SURFACE ACROSS SLICE.
u = AVERAGE PORE WATER PRESSURE ON POTENTIAL SLIDING SURFACE ACROSS SLICE.
R = RADIUS OF MOMENT ARM FOR POTENTIAL SLIDING SURFACE
X = HORIZONTAL DISTANCE FROM CENTROID OF SLICE TO CENTER OF ROTATION.

FIGURE 11
Influence of Stabilizing Pile on Safety Factor



SAFETY FACTOR FOR MOMENT EQUILIBRIUM CONSIDERING THE SAME FORCES AS ABOVE PLUS THE EFFECT OF THE STABILIZING PILE IS EXPRESSED AS:

$$F_s = \frac{\Sigma c' LR + \Sigma (P - uL) R \tan \phi' + TZ}{\Sigma WX}$$

WHERE: T = AVERAGE TOTAL THRUST (PER LIN. FT., HORIZ.) RESISTING SOIL MOVEMENT.

Z = DISTANCE FROM CENTROID OF RESISTING PRESSURE (THRUST) TO CENTER OF ROTATION.

FIGURE 11 (continued)
Influence of Stabilizing Pile on Safety Factor

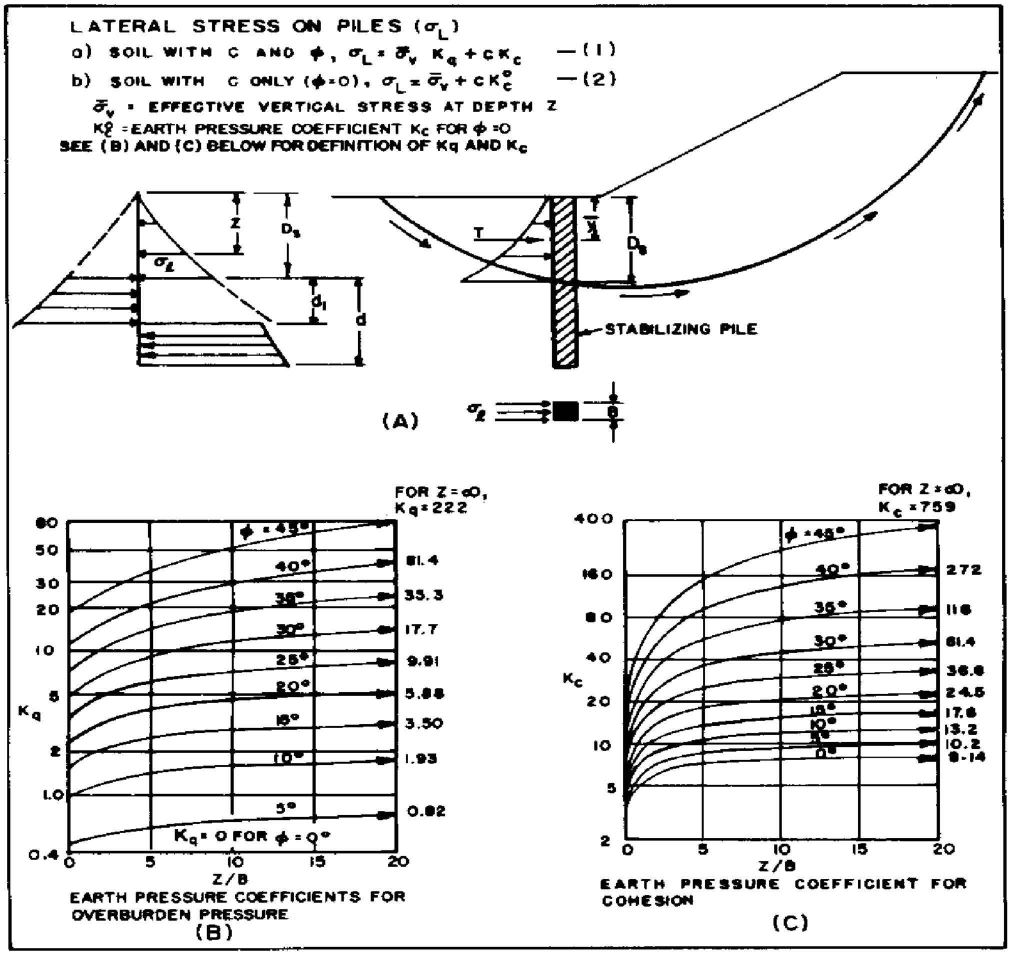


FIGURE 12
Pile Stabilized Slope

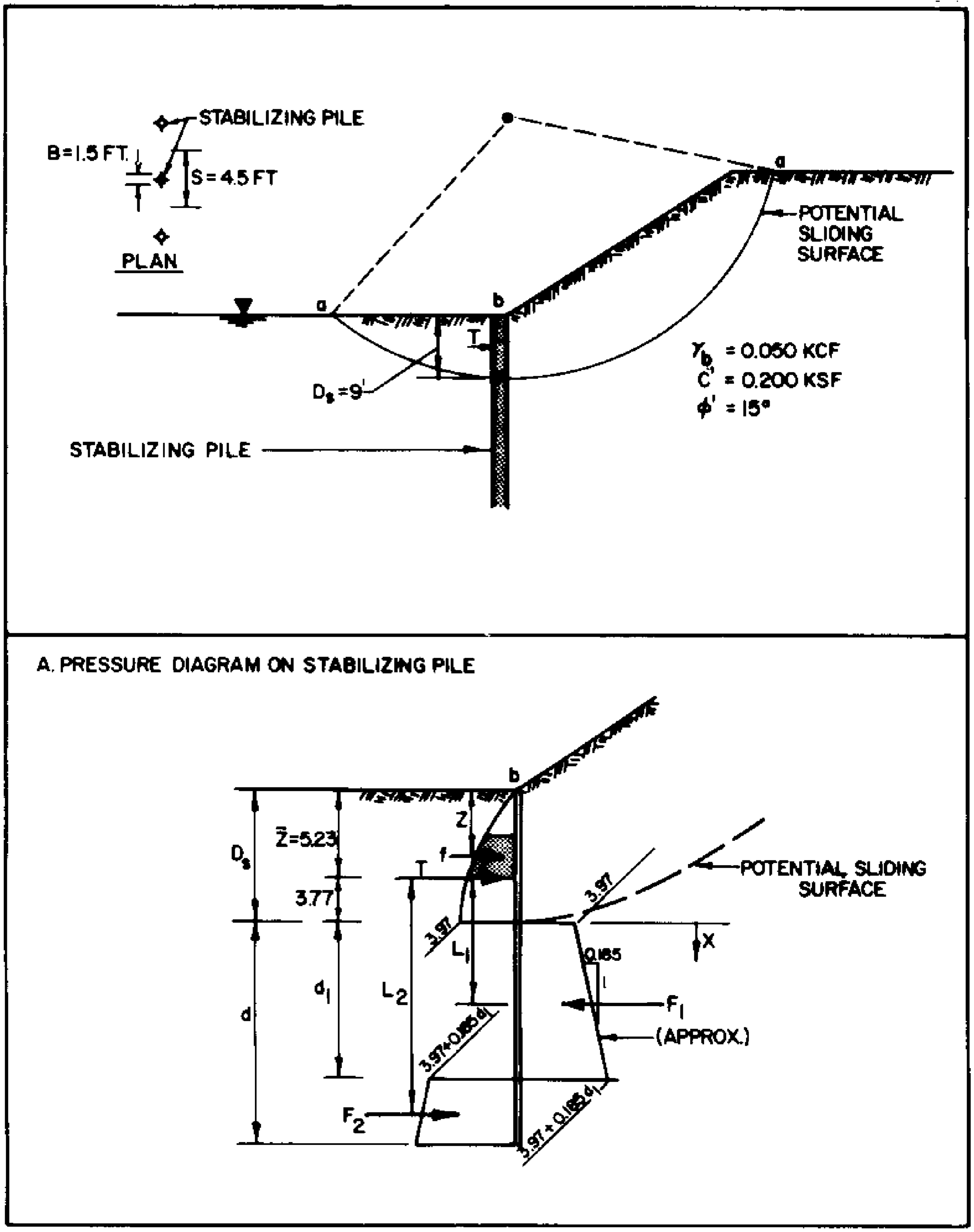


FIGURE 13
Example Calculation - Pile Stabilized Slopes

B. For trial slip surface a-a compute lateral resistance, generated by presence of pile if factor of safety without piles is less than 1.4. Compute pressures using Figure 12.

$\sigma_L = \bar{\sigma}_V K_q + cK_c$ } SEE FIGURE 12 FOR DEFINITIONS

Depth Below Top of Pile z (ft)	z/B	K_q	K_c	Vertical Effective Stress $\bar{\sigma}_V$ (kips/ft^2)	Lateral Resistance to Soil Movement σ_L KSF
0	0	1.5	4.0	0	0.8
3	2	2.1	10.8	0.15	2.48
6	4	2.4	12.8	0.30	3.28
9	6	2.6	14.0	0.45	3.97

C. Compute centroid of lateral resistance (i.e., location of force T)

Depth Range	Resultant Resistance (f) Over Depth Range	z	fz
0-3	$3\left(\frac{0.8 + 2.48}{2}\right) B = 4.92B$	1.5	7.38B
3-6	8.64B	4.5	38.88B
6-9	10.87B	7.5	81.53B
	$\Sigma T = 24.43B$		127.79B

$\bar{z} = 127.79/24.43 = 5.23$ ft

D. Lateral resistance per linear foot of slope

$$T_1 = \Sigma T/S = 24.43 \times 1.5/4.5 = 8.14^k$$

Note that T accounts for three dimensional condition and need not be corrected.

E. Use T_1 in Step D and $\bar{z}$ in Step C to compute additional stabilizing moment for evaluating safety factor including effect of piles (see Figure 11).

FIGURE 13 (continued)
Example Calculation - Pile Stabilized Slopes

F. Compute σ_L at depth corresponding to $Z/B = 20$ ($Z = 30$) in order to compute average Increase of positive resistance with depth:

$$K_q = 3.1, \quad K_c = 16$$

$$\sigma_L = 3.1 \times 30 \times 0.05 + 16 \times 0.2 = 7.85 \text{ KSF}$$

Average increase in lateral resistance below D_s:

$$\sigma_{L_{avg}} = (7.85 - 3.97)/(30 - 9) = 0.185 \text{ KSF/ft}$$

Assume that the direction of lateral resistance changes at depth d_1 beneath failure surface, then:

G. Calculate depth of penetration d by solving the following equations and increase d by 30% for safety:

$$T + F_2 - F_1 = 0 \qquad (1)$$

$$F_1 L_1 = F_2 L_2 \qquad (2)$$

Compute forces per unit pile width:

$$T = 24.43^k$$

$$F_1 = 3.97d_1 + 0.092d_1^2$$

$$F_2 = (3.97 + 0.185d_1)(d-d_1) + 0.092\,(d-d_1)^2$$

$$= 0.092d^2 + 3.97d - 3.97d_1 - 0.092d_1^2$$

H. Use Eq (1) in Step G to calculate d_1 for given values of d.

$$24.43 + 0.092d^2 + 3.97d - 7.94d_1 - 0.185d_1^2 = 0$$

$$d_1^2 + 42.9d_1 - \frac{24.43 + 0.092d^2 + 3.97d}{0.185} = 0$$

Let d = 15.8', then d_1 = 11.0'

From Eq (2) Step G (consider each section of pressure diagram broken down as a rectangle and triangle).

FIGURE 13 (continued)
Example Calculation - Pile Stabilized Slopes

$$F_1 L_1 = \left[3.97 \times 11.0 \times \left(3.77 + \frac{11.0}{2}\right)\right] + \left[\frac{0.185}{2} \times 11.0^2 \times \left(3.77 + \frac{2 \times 11.0}{3}\right)\right]$$

$= 529.1$ FT-KPS

$d - d_1 = 4.8$

$$F_2 L_2 = \left[(3.97 + 0.185 \times 11.0) \times 4.8 \times \left(3.77 + 11.0 + \frac{4.8}{2}\right)\right]$$

$$+ \left[\frac{0.185}{2} \times 4.8^2 \times \left(3.77 + 11.0 + \frac{2 \times 4.8}{3}\right)\right]$$

$= 533.2$

$F_1 L_1 - F_2 L_2 = -4.1$

$d \approx 15.8'$ O.K.

I. Design

Increase d by 30% to obtain the practical driving depth

$d = 15.8 \times 1.3 = 20.5'$

LOCATE POINT OF ZERO SHEAR

$24.43 = 3.97X + 0.092X^2$

$X^2 + 43.15X - 265.54 = 0$

$$X = \frac{-43.15 \pm \sqrt{43.15^2 + 4 \times 265.54}}{2}$$

$= 5.46'$

COMPUTE MAXIMUM BENDING ON PILE (B=1.5')

$$M_{max} = \left[24.43 \times (3.77 + 5.46) - \left(\frac{3.97 \times 5.46^2}{2} + \frac{0.185 \times 5.46^3}{2 \times 3}\right)\right] \times 1.5$$

$= 241.9$ Kp-FT

CHECK PILE SECTION VS M_{max}

NOTES:

a. Higher embedment may be required to minimize slope movements.

b. Use residual shear strength parameters if appropriate.

c. Analysis applicable for safety factor $\leq$ 1.4 without piles. Soil movement assumed to be large enough to justify assumption on rupture conditions.

FIGURE 13 (continued)
Example Calculation - Pile Stabilized Slopes

2. TYPES OF PROTECTION AVAILABLE. The usual protection against erosion by wind and rainfall is a layer of rock, cobbles, or sod. Protection from wave action may be provided by rock riprap (either dry dumped or hand placed), concrete pavement, precast concrete blocks, soil-cement, fabric, and wood. See Table 8, Chapter 6 for additional guidance.

a. Stone Cover. A rock or cobbles cover of 12" thickness is sufficient to protect against wind and rain.

b. Sod. Grasses suitable for a given locality should be selected with provision for fertilizing and uniform watering.

c. Dumped Rock Riprap. This provides the best protection against wave action. It consists of rock fragments dumped on a properly graded filter. Rock used should be hard, dense, and durable against weathering and also heavy enough to resist displacement by wave action. See Table 5 for design guidelines. For additional design criteria see Figure 14, Chapter 6.

d. Hand-placed Riprap. Riprap is carefully laid with minimum amount of voids and a relatively smooth top surface. Thickness should be one-half of the dumped rock riprap but not less than 12". A filter blanket must be provided and enough openings should be left in the riprap facing to permit easy flow of water into or out of the riprap.

e. Concrete Paving. As a successful protection against wave action concrete paving should be monolithic and of high durability. Underlying materials should be pervious to prevent development of uplift water pressure. Use a minimum thickness of 6".

When monolithic construction is not possible, keep the joints to a minimum and sealed. Reinforce the slab at mid depth in both directions with continuous reinforcement through the construction joints. Use steel area in each direction equal to 0.5% of the concrete area.

f. Gabions. Slopes can be protected by gabions. Use of these is discussed in DM-7.02, Chapter 3.

TABLE 5
Thickness and Gradation Limits of Dumped Riprap

Slope	Nominal thickness inches	Maximum Size	Gradation, percentage of stones of various weights, pounds[1]		
			40 to 50 percent greater than	50 to 60 percent from - to	0 to 10 percent less than[2]
3:1	30	2,500	1,250	75 - 1,250	75
2:1	36	4,500	2,250	100 - 2,250	100

[1] Sand and rock dust shall be less than 5 percent, by weight, of the total riprap material.

[2] The percentage of this size material shall not exceed an amount which will fill the voids in larger rock.

REFERENCES

1. Transportation Research Board, Landslide Analysis and Control, Special Report 176, 1978.

2. Lambe, T. W. and Whitman, R. V., Soil Mechanics, John Wiley & Sons, Inc., 1969.

3. Fang, H. Y., Stability of Earth Slopes, Foundation Engineering Handbook, Winterkorn and Fang, ed., Chapter 10, Van Nostrand Reinhold Co., New York, 1975.

4. Janbu, N., Stability Analysis of Slopes with Dimensionless Parameters, Harvard Soil Mechanics Series No. 46, Harvard University, Cambridge, MA.

5. Jakobsen, R. E., The Design of Embankments on Soft Clays, Geotechnique, 1948.

6. Von Thun, J. L., The Practical and Realistic Solution of Rock Slope Stability Problems, Design Methods in Rock Mechanics, Proceedings of Sixteenth Symposium on Rock Mechanics, ASCE, September 22-24, 1977.

7. Sarma, S. K. and Bhave, M. V., Critical Acceleration Versus Static Factor of Safety in Stability Analysis of Earth Dams and Embankments, Geotechnique, Vol. 24, No. 4, 1974.

8. Newmark, N. M., Effects of Earthquakes on Dams and Embankments, Geotechnique, Vol. 15, No. 2, 1965.

9. Duncan, J. M. and Buchignani, A. L., An Engineering Manual for Slope Stability Studies, Department of Civil Engineering, Institute of Transportation and Traffic Engineering, University of California, Berkeley, March, 1975.

10. DeBeer, E. E. and Wallays, M., Forces Induced in Piles by Unsymmetrical Surcharges on the Soil Around the Pile, Proceedings of Fifth European Conference Soil Mechanics and Foundation Engineering, Madrid, Volume 1, p. 335, 1972.

BIBLIOGRAPHY

American-Society of Photogrammetry, Manual of Photo Interpretation, Washington, D. C., 1960.

ASTM STP 447, Determination of the In Situ Modulus of Deformation of Rock, Symposium, Denver, February 2-7, 1969.

Ash, J. L. et al., Improved Subsurface Investigation for Highway Tunnel Design and Construction, Vol. 1, Subsurface Investigation System Planning, FHWA, May 1974.

Burmister, D. M., Physical, Stress-Strain, and Strength Responses of Granular Soils, ASTM Spec. Pub. No. 322, ASTM, Philadelphia, pp 67-97, 1962.

Cedergren, H., Seepage, Drainage and Flow Nets, McGraw-Hill Book Co., Inc., New York, 1967.

Chen, W. F., Limit Analysis and Soil Plasticity, Elsevier Scientific Publishing Co., New York, 1975.

Dallaire, G., Controlling Erosion and Sedimentation at Construction Sites, Civil Engineering, ASCE, pp 73-77, 1976.

Foster, C. R. and Ahlvin, P. G., Stresses and Deflections Induced by Uniform Circular Load, Highway Research Board Proceedings, Highway Research Board, Washington, D. C., 1954.

Harr, M. E., Groundwater and Seepage, McGraw-Hill Book Co., Inc., New York, 1962.

Hoek, E. and Bray, J. W., Rock Slope Engineering, 2nd Edition, The Institute of Mining and Metallurgy, London, 1977.

Menard, L. F., Interpretation and Application of Pressuremeter Test Results, Soils-Soils, Vol. 26, pp 1-43, 1975.

Mooney, H. M., Handbook of Engineering Seismology, Bison Instruments, Inc., Minneapolis, MN., 1973.

Noorany, I. and Gizienski, S. F., Engineering Properties of Submarine Soils, A State-of-the-Art Review, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM5, 1970.

Peck, R. B., Stability of Natural Slopes, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM4, 1967.

Poulos, N. C. and Davis,E. H., Elastic Solutions for Soil and Rock Mechanics, John Wiley & Sons, Inc., New York, 1974.

Seed, H. B., A Method for Earthquake Resistant Design of Earth Dams, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 92, No. SM1, 1966.

Seed, H. B., Makdisi, F. I. and De Alba, P., Performance of Earthdams During Earthquakes, Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT7, 1978.

Skempton, A. W., and Hutchinson, J., Stability of Natural Slopes and Embankment Foundations, State-of-the-Art Paper, Proceedings, Seventh International Conference Soil Mechanics Foundation Engineering, Mexico, pp 291-340, 1969.

Soil Conservation Service, USDA, A Comprehensive System, 7th Approximation, U.S. Government Printing Office, Washington, D. C. p. 265, 1960, and supplement 1967, 1968, and 1970.

Sowers, C. F., Shallow Foundations, Foundation Engineering, Leonards, Editor, McGraw-Hill Book Co., Inc., New York, Chapter 6, 1962.

Stagg, K. G., Insitu Tests on the Rock Mass, Rock Mechanics in Engineering Practice, Stagg and Zienkiewicz, Editors, John Wiley and Sons, New York, Chapter 5, 1969.

Terzaghi, K., Varieties of Submarine Slope Failures, Proceedings, Eighth Texas Conference on Soil Mechanics and Foundation Engineering, 1956.

Terzaghi, K., Stability of Steep Slopes on Hard, Unweathered Rock, Geotechnique, Vol. 12, No. 4, 1962.

Transportation Research Record 581: Innovations in Subsurface Exploration of Soils, Transportation Research Board, 1976.

Underwood, L. B., Exploration and Geologic Prediction for Underground Works, Subsurface Exploration for Underground Excavation and Heavy Construction, ASCE, New York, pp 65-83, 1974.

U.S.G.S., Earth Science Information in Land-Use Planning -Guidelines for Earth Scientists and Planners, Geological Survey Circular No. 721, U. S. Geological Survey, Fairfax, VA., 1976.

APPENDIX A
Listing of Computer Programs

Subject	Program	Description	Availability
Field Exploration, Testing and Instrumentation (Chapter 2)	SOILS 1	Geotechnical data file for Navy facilities.	Naval Facilities Engineering Command HQ, Alexandria, VA
	SOILS 2	Retrieval of data from SOILS 1.	"
	SOILS 3	Modification or addition to existing data base file.	"
Distribution of Stresses (Chapter 4) and Settlement Analyses (Chapter 5)	HSPACE GESA Catalog No. E01-0002-00030	Stresses and displacements in an elastic half-space with interior loads.	Geotechnical Engineering Software Activity University of Colorado Boulder, CO 80309 (GESA)
	SAS3D GESA Catalog No. E01-0003-00042	Stresses and displacements in an isotropic elastic half-space due to rectangular surface loads.	"
	CANDE	Analysis for design of buried conduits. Solution methods include closed form elastic methods and a general finite element solution.	Federal Highway Administration, Office of Research and Development Washington, D.C.
	ICES SEPOL 1	Analysis of stress distribution, magnitude and rate of settlement for horizontally layered soil, and multiple complex surface loads. Stresses calculated assuming homogeneous elastic half-space.	ICES Users Group, Inc. ICES Distribution Agency P.O. Box 142, MIT Branch Cambridge, MA 02139

APPENDIX A (continued)
Listing of Computer Programs

Subject	Program	Description	Availability
Distribution of Stress (Chapter 4) and Settlement Analyses (Chapter 5)	FEECON	A finite element analysis for computing undrained deformation of soft clay foundations under granular embankments. Stresses calculated can be used to evaluate yield conditions and stability.	Massachusetts Institute of Technology (MIT) Cambridge, MA
	PROGRS GESA Catalog No. E02-0002-00014	One dimensional consolidation of multi-layered system using the finite difference method.	Geotechnical Engineering Software Activity
	CONS-2DFE	Finite element program for solving consolidation problem under plane strain (or axisymmetric) conditions.	Virginia Polytechnic Institute and State University, Blacksburg, VA 24061
	SDRAIN GESA Catalog No. E02-0003-00017	One dimensional settlement analysis and excess pore pressure due to embankments on layered soils using sand drains.	Geotechnical Engineering Software Activity
	SSTIN-2DFE	Finite element program for two dimensional (plane strain or axisymmetric) soil structure interaction problems.	Virginia Polytechnic Institute and State University
Seepage and Drainage (Chapter 6)	FEDAR GESA Catalog No. E07-NOQA-0050	Finite element program for analysis of steady confined and unconfined seepage.	Geotechnical Engineering Software Activity or Massachusetts Institute of Technology

GLOSSARY

Activity of Clay - The ratio of plasticity index to percent by weight of the total sample that is smallar than 0.002 mm in grain size. This property is correlated with the type of clay material.

Anisotropic Soil - A soil mass having different properties in different directions at any given point referring primarily to stress-strain or permeability characteristics.

Capillary Stresses - Pore water pressures less than atmospheric values produced by surface tension of pore water acting on the meniscus formed in void spaces between soil particles.

Clay Size Fraction - That portion of the soil which is finer than 0.002 mm, not a positive measure of the plasticity of the material or its characteristics as a clay.

Desiccation - The process of shrinkage or consolidation of the fine-grained soil produced by increase of effective stresses in the grain skeleton accompanying the development of capillary stresses in the pore water.

Effective Stress - The net stress across points of contact of soil particles, generally considered as equivalent to the total stress minus the pore water pressure.

Equivalent Fluid Pressure - Horizontal pressures of soil, or soil and water, in combination, which increase linearly with depth and are equivalent to those that would be produced by a heavy fluid of a selected unit weight.

Excess Pore Pressures - That increment of pore water pressures greater than hydro-static values, produced by consolidation stresses in compressible materials or by shear strain.

Exit Gradient - The hydraulic gradient (difference in piezometric levels at two points divided by the distance between them) near to an exposed surface through which seepage is moving.

Flow Slide - Shear failure in which a soil mass moves over a relatively long distance in a fluidlike manner, occurring rapidly on flat slopes in loose, saturated, uniform sands, or in highly sensitive clays.

Hydrostatic Pore Pressures - Pore water pressures or groundwater pressures exerted under conditions of no flow where the magnitude of pore pressures increase linearly with depth below the ground surface.

Isotropic Soil - A soil mass having essentially the same properties in all directions at any given point, referring directions at any given point, referring primarily to stress-strain or permeability characteristics.

Normal Consolidation - The condition that exists if a soil deposit has never been subjected to an effective stress greater than the existing overburden pressure and if the deposit is completely consolidated under the existing overburden pressure.

Overconsolidation - The condition that exists if a soil deposit has been subjected to an effective stress greater than the existing overburden pressure.

Piezometer - A device installed for measuring the pressure head of pore water at a specific point within the soil mass.

Piping - The movement of soil particles as the result of unbalanced seepage forces produced by percolating water, leading to the development of boils or erosion channels.

Plastic Equilibrium - The state of stress of a soil mass that has been loaded and deformed to such an extent that its ultimate shearing resistance is mobilized at one or more points.

Positive Cutoff - The provision of a line of tight sheeting or a barrier of impervious material extending downward to an essentially impervious lower boundary to intercept completely the path of subsurface seepage.

Primary Consolidation - The compression of the soil under load that occurs while excess pore pressures dissipate with time.

Rippability - The characteristic of dense and rocky soils that can be excavated without blasting after ripping with a rock rake or ripper.

Slickensides - Surfaces with a soil mass which have been smoothed and striated by shear movements on these surfaces.

Standard Penetration Resistance - The number of blows of a 140-pound hammer, falling 30 inches, required to advance a 2-inch O.D., split barrel sampler 12 inches through a soil mass.

Total Stress - At a given point in a soil mass the sum of the net stress across contact points of soil particles (effective stress) plus the pore water pressure at the point.

Underconsolidation - The condition that exists if a soil deposit is not fully consolidated under the existing overburden pressure and excess hydrostatic pore pressures exist within the material.

Varved Silt or Clay - A fine-grained glacial lake deposit with alternating thin layers of silt or fine sand and clay, formed by variations in sedimentation from winter to summer during the year.

SYMBOLS

Symbol	Designation
A	Cross-sectional area.
A+c,	Activity of fine-grained soil.
a+v,	Coefficient of compressibility.
B,b	Width in general; or narrow dimension of a foundation unit.
CBR	California Bearing Ratio.
C+c,	Compression index for virgin consolidation.
CD	Consolidated-drained shear test.
C+r,	Recompression index in reconsolidation.
C+s,	Swelling index.
CU	Consolidated-undrained shear test.
C+u,	Coefficient of uniformity of grain size curve.
C+z,	Coefficient of curvation of gradation curve.
C+[alpha],	Coefficient of secondary compression.
c	Cohesion intercept for Mohr's envelop of shear strength based on total stresses.
c'	Cohesion intercept for Mohr's envelope of shear strength based on effective stresses.
c+h,	Horizontal coefficient of consolidation.
c+v,	Vertical coefficient of consolidation.
D,d	Depth, diameter, or distance.
D+r,	Relative density.
D+10,	Effective grain size of soil sample; 10% by dry weight of sample is smaller than this grain size.
D+5,, D+60, D+85,	Grain size division of a soil sample, percent of dry weight smaller than this grain size is indicated by subscript.
E	Modulus of elasticity of structural material.
E+s,	Modulus of elasticity or "modulus of deformation" of soil.
e	Void ratio.
e+f,	Final void ratio reached in loading phase of consolidation test.
e+o,	Initial void ratio in consolidation test generally equal to natural void in situ.
e+r,	Void ratio existing at the start of rebound in a consolidation test.
F	Shape factor describing the characteristics of the flow field in underseepage analysis.
F+s,	Safety factor in stability or shear strength analysis.
G	Specific gravity of solid particles in soil sample, or shear modulus of soil.
H,h	In general, height or thickness. For

analysis of time rate of consolidation, H is the maximum vertical dimension of the drainage path for pore water.

h+c, Capillary head formed by surface tension in pore water.

H+t, Depth of tension cracks or total thickness of consolidating stratum or depth used in computing loads on tunnels.

H+w, Height of groundwater or of open water above a base level.

I Influence value for vertical stress produced by superimposed load, equals ratio of stresses at a point in the foundation to intensity of applied load.

Symbol	Designation
i	Gradient of groundwater pressures in underseepage analysis.
K+A,	Coefficient of active earth pressures.
K+p,	Coefficient of passive earth pressures.
K+v,	Modulus of subgrade reaction for bearing plate or foundation of width b.
K+v*,	Modulus of subgrade reaction for 1 ft square bearing plate at ground surface.
k	Coefficient of permeability in general.
k+H,	Coefficient of permeability in horizontal direction.
k+m,	Mean coefficient of permeability of anisotropic subsoil.
ksf	Kips per sq ft pressure intensity.
ksi	Kips per sq in pressure intensity.
k+V,	Coefficient of permeability in vertical direction.
L,1	Length in general or longest dimension of foundation unit.
LI	Liquidity index.
LL	Liquid limit.
m+v,	Coefficient of volume compressibility in consolidation test.
n	Porosity of soil sample.
n+d,	Number of equipotential drops in flow net analysis of underseepage.
n+e,	Effective porosity, percent by volume of water drainable by gravity in total volume of soil sample.
n+f,	Number of flow paths in flow net analysis of underseepage.
OMC	Optimum moisture content of compacted soil.
P+A,	Resultant active earth force.
P+AH,	Component of resultant active force in horizontal direction.
pcf	Density in pounds per cubic foot.
P+c,	Preconsolidation stress.
P+h,	Resultant horizontal earth force.
P+o,	Existing effective overburden pressure acting at a specific height in the soil profile or on a soil sample.
PI	Plasticity index.
PL	Plastic limit.
P+P,	Resultant passive earth force.
P+PH,	Component of resultant passive earth force in horizontal direction.
P+v,	Resultant vertical earth force.
P+w,	Resultant force of water pressure.
p	Intensity of applied load.
q	Intensity of vertical load applied to foundation unit.
q+u,	Unconfined compressive strength of soil sample.
q+ult,	Ultimate bearing capacity that causes

	shear failure of foundation unit.
R,r	Radius of pile, caisson well or other right circular cylinder.
R+o,	Radius of influence of a well, distance from the well along a radial line to the point where initial groundwater level is unaltered.
r+e,	Effective radius of sand drain.
r+s,	Radius of smear zone surrounding sand drain.
r+w,	Actual radius of sand drain.
S	Percent saturation of soil mass.
SI	Shrinkage index.

Symbol	Designation
SL	Shrinkage limit.
S+t,	Sensitivity of soil, equals ratio of remolded to undisturbed shear strength.
s	Shear strength of soil for a specific stress or condition in situ, used instead of strengh parameters c and [phi].
T+o,	Time factor for time at end of construction in consolidation analysis for gradual loading.
T+v,	Time factor in consolidation analysis for instantaneous load application.
tsf	Tons per sq ft pressure intensity.
t,t+1,, t+2,,t+n,	Time intervals from start of loading to the points 1, 2, or n.
t+50,,t+100,	Time required for a percent consolidation to be completed indicated by subscript
U	Resultant force of pore water or groundwater pressures acting on a specific surface within the subsoils.
U	Average degree of consolidation at any time.
u	Intensity of pore water pressure.
UU	Unconsolidated-undrained shear test.
V+a,	Volume of air or gas in a unit total volume of soil mass.
V+s,	Volume of solids in a unit total volume of soil mass.
V+v,	Volume of voids in a unit total volume of soil mass.
V+w,	Volume of water in a unit total volume of soil mass.
W+s,	Weight of solids in a soil mass or soil sample.
W+t,	Total weight of soil mass or soil sample.
W+w,	Weight of water in a soil mass or soil sample.
w	Moisture content of soil.
[gamma]+D,	Dry unit weight of soil
[gamma]+MAX,	Maximum dry unit weight of soil determined from moisture content dry unit weight curve.
[gamma]+SAT,	Saturated unit weight of soil.
[gamma]+SUB,,[gamma]+b,	Submerged (buoyant) unit weight of soil mass.
[gamma]+T,	Wet unit weight of soil above the groundwater table.
[gamma]+W,	Unit weight of water, varying from 62.4 pcf for fresh water to 64 pcf for sea water.
[epsilon]	Unit strain in general.
[epsilon]+a,	Axial strain in triaxial shear test.
[W-DELTA]e	Change in void ratio corresponding to a change in effective stress,

	[W-DELTA]p.
[delta], [delta]+v,, [delta]+c,	Magnitude of settlement for various conditions.
[phi]	Angle of internal friction or "angle of shearing resistance," obtained from Mohr's failure envelope for shear strength.
[sigma]*	Total major principal stress.
[sigma]+3,	Total minor principal stress
)	
[sigma]*.	Effective major principal stress
)	
[sigma]+3,	Effective minor principal stress.
[sigma]+x,, [sigma]+y,, [sigma]+z,	Normal stresses in coordinate directions.
[tau]	Intensity of shear stress.
[tau]+MAX,	Intensity of maximum shear stress.
[upsilon]	Poisson's Ratio

INDEX

A

B

C

D

E

S

T

```
+))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))),
* CCB Application Notes:                                                 *
*                                                                        *
* 1. Character(s) preceded & followed by these symbols (. -) or (+ ,)   *
*    are super- or subscripted, respectively.                            *
*    EXAMPLES:  42m.3-  =  42 cubic meters                               *
*               CO+2,   =  carbon dioxide                                *
*                                                                        *
* 2. All degree symbols have been replaced with the word deg.            *
*                                                                        *
* 3. All plus or minus symbols have been replaced with the symbol +/-.  *
*                                                                        *
* 4. All table note letters and numbers have been enclosed in square    *
*    brackets in both the table and below the table.                     *
*                                                                        *
* 5. Whenever possible, mathematical symbols have been replaced with    *
*    their proper name and enclosed in square brackets.                  *
.)))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))))-
```

Naval Facilities Engineering Command
200 Stovall Street
Alexandria, Virginia 22332-2300 APPROVED FOR PUBLIC RELEASE

Foundations &
Earth Structures

DESIGN MANUAL 7.02
REVALIDATED BY CHANGE 1 SEPTEMBER 1986

NAVAL FACILITIES ENGINEERING COMMAND
PUBLICATIONS TRANSMITTAL

SN 0525-LP-300-7071

---

1. TITLE	NUMBER	DATE
DM-7.02 FOUNDATIONS ANTD EARTH STRUCTURES	Change 1	September 1986

---

2. ATTACHMENTS

New Cover.
New Record of Document Changes page.
New pages iii through xiv.
New pages 7.2-35 through 7.2-38.
New pages 7.2-47 through 7.2-50.
New pages 7.2-53 and 7.2-54.
New pages 7.2-57 through 7.2-64.
New pages 7.2-127 through 7.2-130.
New pages 7.2-151 and 7.2-152.
New pages 7.2-175 through 7.2-178.
New pages 7.2-243 and 7.2-244.
New DD Form 1426.

---

3. EXPLANATION

Remove old cover dated May 1982 and replace with attached new cover dated September 1986.
Insert new Record of Document Changes page immediately after new cover page.
Remove paged 7.2-iii through 7.2-xvii and replace with attached new pages iii through xiv.
Remove pages 7.2-35 through 7.2-38 and replace with attached new pages 7.2-35 through 7.2-38.
Remove pages 7.2-47 through 7.2-50 and replace with attached new pages 7.2-47 through 7.2-50.
Remove pages 7.2-53 and 7.2-54 and replace with attached new pages 7.2-53 and 7.2-54.
Remove pages 7.2-57 through 7.2-64 and replace with attached new pages 7.2-57 through 7.2-64.
Remove pages 7.2-127 through 7.2-130 and replace with attached new pages 7.2-127 through 7.2-130.
Remove pages 7.2-151 and 7.2-152 and replace with attached new pages 7.2-151 and 7.2-152.
Remove pages 7.2-175 through 7.2-173 and replace with attached new pages 7.2-175 through 7.2-178.
Remove pages 7.2-243 and 7.2-244 and replace with attached new pages 7.2-243 and 7.2-244.
Insert new DD Form 1426 in back of manual.

DM-7.02 FOUNDATIONS AND EARTH STRUCTURES is revalidated for three years after incorporation of this change.

---

4. DISTRIBUTION

See reverse.

For sale by the Superintendent of Documents, U.S. Government Printing

Office, Washington, D.C. 20402

NAVFAC DM/P MANUAL DISTRIBUTION (01/87):
(1 copy each unless otherwise specified)

SNDL	
39B	COMCB
49	ADMSUPPU
A2A	NAVY STAFF (ONR only)
A3	CHIEF OF NAVAL OPERATIONS
A5	NAVPERS & BUMED
A6	COMMANDANT MC (code LFF)
B2A	(JCS, NSA, DLA, DNA only)
B5	USCG only)
E3A	LAB ONR(Wash.DC only)
FA6	NAVAIRSTA LANT
FA7	NAVSTA LANT
FA10	SUB BASE LANT
FA18	PHIBASE LANT
FA23	NAVFAC LANT
FA32	CINCLANT CBU
FB6	NAVAIRFAC PAC
FB7	NAVAIRSTA PAC
FB10	NAVSTA PAC (Seattle only)
FB13	SUBASE PAC (Bangor only)
FB21	PHIBASE PAC
FB34	FLEACTPAC (Kadena,Sasebo only)
FB36	FAC PAC
FB48	SUPPFAC PAC
FC3	ACTIVITY EUR (London only)
FC5	SUPPORT ACTIVITY EUR
FC7	NAVAL STATION EUR
FC12	FLEET SUPPORT OFFICE
FC14	NAVAIRSTA EUR
FD1	OCEANOGRAPHY COMM
FE1	SECURITY GROUP HQ
FE2	SECURITY STATION
FE4	SECGRUACT (Edzell, Hanza Homestead, Sabana Seca, Sonoma, Winter Harbor only)
FF6	NAVAL OBSERVATORY
FF38	NAVAL ACADEMY
FG1	TELECCOMMCOM
FG2	COMSTA (Balboa, Harold Holt, Nea Makri, Thurso, Stockton, Yokosuka, San Miguel only)
FG3	COMM UNIT (East Machias only)
FG6	CA MASTER STAS (Norfolk only)
FH1	COMNAVMEDCOM
FKA1A	SYSCOM, AIR
FKA1B	SYSCOM, ELEX
FKA1C	NAVFAC,(Code 04M2, 30 copies)
FKA1F	SYSCOM, SUP
FKA1G	SYSCOM, SEA
FKM8	SUPPLY ANNEX
FKM9	NSC (Oakland only)
FKM13	SHIPS PARTS & CONTROL CEN
FKM15	ASO (Philadelphia only)
FKN1	LANTDIV (100 copies) CHESDIV (50 copies) NORTHDIV (75 copies) PACDIV, SOUTHDIV, WESTDIV, (200 copies each)
FKN2	CBC

FKN3	OICC (6 copies each)
FKN5	PWC (5 copies each)
FKN7	ENERGY ENVIRON SUPP ACT
FKN10	SUPPORT ACT, NAV FAC
FKN11	CIVIL ENGINEERING LAB
FKP1B	WEAPONS STATION
FKP1E	UNDERSEA WARFARE ENGR STA
FKP1J	NAVAL ORDNANCE STATION
FKP1M	WEAPONS SUPPLY CENTER
FKP7	NAVAL SHIPYARDS
FKQ3A	ELECTRONIC STMS ENGR CEN
FKQ6A	AIR DEVELOPMENT CENTER
FKQ6B	COASTAL SYSTEMS CENTER
FKQ6C	OCEAN SYSTEMS CENTER
FKQ6E	SHIP RESEARCH & DEV.CEN.
FKQ6F	SURFACE WEAPONS CENTER
FKQ6G	UNDERWATER SYSTEMS CENTER
FKQ6H	NAVAL WEAPONS CENTER
FKR1A	AIR STATIONS R & D
FKR1B	AIR REWORK FACILITY
FKR3A	AIR ENGINEERING CENTER
FKR3H	AIR PROPULSION CENTER
FKR4B	MISSILE RANGE FACILITY
FKR5	AVIONICS CENTER
FKR7E	AVIATION LOGISTICS CENTER
FR3	AIR STATIONS
FR4	AIR FACILITIES
FR15	SUPPACT
FT1	CHIEF NAVAL ENGR. TRAINING
FT2	CHIEF NAVAL AIR TRAINING
FT6	AIR STATIONS CNET
FT13	AIR TECH TRAINING CEN
FT18	CONSTR. BATTALION UNIT CBC
FT19	ADMINCOM (San Diego only)
FT22	FLECOMBATRACEN (Va. Beach only)
FT28	EDUCATION & TRAINING CEN
FT31	TRAINING CENTER
FT37	SCHOOL CEC OFFICERS
FT55	SCHOOL SUPPLY CORPS
FT78	ED & TRNG PGM DEV CEN
V2	MARCORPS BARRACKS SUPPORT
V3	MC AIR BASE COMMAND
V5	MCAS (less Beaufort)
V5	MCAS (Beaufort only,5 cys)
V8	MC RECRUIT DEPOT
V12	MC DEVELOP.& EDUCATION CEN
V14	MC HEADQUARTERS BATTALION
V15	MC DISTRICT
V17	MC CAMP
V23	MC LOGISTICS BASE
V25	MC AIR GROUND COMBAT CEN

REMAINDER TO: RECEIVING OFFICER
Naval Publications and Forms Center
5801 Tabor Avenue, Phila., PA 19120
FOR IMMEDIATE STOCK

Information on how to obtain additional copies is contained in References.

NAVAL FACILITIES ENGINEERING COMMAND
PUBLICATIONS TRANSMITTAL

SN 0525-LP-300-7071
))

1. TITLE	NUMBER	DATE
DM-7.02 FOUNDATIONS ANTD EARTH STRUCTURES	Change 1	September 1986

))
2. ATTACHMENTS

New Cover.
New Record of Document Changes page.
New pages iii through xiv.
New pages 7.2-35 through 7.2-38.
New pages 7.2-47 through 7.2-50.
New pages 7.2-53 and 7.2-54.
New pages 7.2-57 through 7.2-64.
New pages 7.2-127 through 7.2-130.
New pages 7.2-151 and 7.2-152.
New pages 7.2-175 through 7.2-178.
New pages 7.2-243 and 7.2-244.
New DD Form 1426.
))
3. EXPLANATION

Remove old cover dated May 1982 and replace with attached new cover dated September 1986.
Insert new Record of Document Changes page immediately after new cover page.
Remove paged 7.2-iii through 7.2-xvii and replace with attached new pages iii through xiv.
Remove pages 7.2-35 through 7.2-38 and replace with attached new pages 7.2-35 through 7.2-38.
Remove pages 7.2-47 through 7.2-50 and replace with attached new pages 7.2-47 through 7.2-50.
Remove pages 7.2-53 and 7.2-54 and replace with attached new pages 7.2-53 and 7.2-54.
Remove pages 7.2-57 through 7.2-64 and replace with attached next pages
7.2-57 through 7.2-64.
Remove pages 7.2-127 through 7.2-130 and replace with attached new pages 7.2-127 through 7.2-130.
Remove pages 7.2-151 and 7.2-152 and replace with attached new pages 7.2-151 and 7.2-152.
Remove pages 7.2-175 through 7.2-173 and replace with attached new pages 7.2-175 through 7.2-178.
Remove pages 7.2-243 and 7.2-244 and replace with attached new pages 7.2-243 and 7.2-244.
Insert new DD Form 1426 in back of manual.

DM-7.02 FOUNDATIONS AND EARTH STRUCTURES is revalidated for three years after incorporation of this change.
))
4. DISTRIBUTION

See reverse.

NAVFAC DM/P MANUAL DISTRIBUTION (01/87):
(1 copy each unless otherwise specified)

SNDL	
39B	COMCB
49	ADMSUPPU
A2A	NAVY STAFF (ONR only)
A3	CHIEF OF NAVAL OPERATIONS
A5	NAVPERS & BUMED
A6	COMMANDANT MC (code LFF)
B2A	(JCS, NSA, DLA, DNA only)
B5	USCG only)
E3A	LAB ONR(Wash.DC only)
FA6	NAVAIRSTA LANT
FA7	NAVSTA LANT
FAIO	SUB BASE LANT
FA18	PHIBASE LANT
FA23	NAVFAC LANT
FA32	CINCLANT CBU
FB6	NAVAIRFAC PAC
FB7	NAVAIRSTA PAC
FB10	NAVSTA PAC (Seattle only)
FB13	SUBASE PAC (Bangor only)
FB21	PHIBASE PAC
FB34	FLEACTPAC (Kadena,Sasebo only)
FB36	FAC PAC
FB48	SUPPFAC PAC
FC3	ACTIVITY EUR (London only)
FC5	SUPPORT ACTIVITY EUR
FC7	NAVAL STATION EUR
FC12	FLEET SUPPORT OFFICE
FC14	NAVAIRSTA EUR
FDl	OCEANOGRAPHY COMM
FE1	SECURITY GROUP HQ
FE2	SECURITY STATION
FE4	SECGRUACT (Edzell, Hanza Homestead, Sabana Seca, Sonoma, Winter Harbor only)
FF6	NAVAL OBSERVATORY
FF38	NAVAL ACADEMY
FG1	TELECOMMCOM
FG2	COMSTA (Balboa, Harold Holt, Nea Makri, Thurso, Stockton, Yokosuka, San miguel only)
FG3	COMM UNIT (East Machias only)
FG6	CA MASTER STAS (Norfolk only)
FH1	COMNAVMEDCOM
FKA1A	SYSCOM, AIR
FKA1B	SYSCOM, ELEX
FKA1C	NAVFAC, (Code 04M2, 30 copies)
FKA1F	SYSCOM, SUP
FKA1G	SYSCOM, SEA
FKM8	SUPPLY ANNEX
FKM9	NSC (Oakland only)
FKM13	SHIPS PARTS & CONTROL CEN
FKM15	ASO (Philadelphia only)
FKN1	LANTDIV (100 copies) CHESDIV (50 copies) NORTHDIV (75 copies) PACDIV, SOUIHDIV, WESTDIV, (200 copies each)
FKN2	CBC

FKN3	OICC (6 copies each)
FKN5	PWC (5 copies each)
FKN7	ENERGY ENVIRON SUPP ACT
FKN10	SUPPORT ACT NAV FAC
FKN11	CIVIL ENGINEERING LAB
FKP1B	WEAPONS STATION
FKP1E	UNDERSEA WARFARE ENGR STA
FKP1J	NAVAL ORDNANCE STATION
FKP1M	WEAPONS SUPPLY CENTER
FKP7	NAVAL SHIPYARDS
FKQ3A	ELECTRONIC STMS ENGR CEN
FKQ6A	AIR DEVELOPMENT CENTER
FKQ6B	COASTAL SYSTEMS CENTER
FKQ6C	OCEAN SYSTEMS CENTER
FKQ6E	SHIP RESEARCH & DEV.CEN,
FKQ6F	SURFACE WEAPONS CENTER
FKQ6G	UNDERWATER SYSTEMS CEN'TER
FKQ6H	NAVAL WEAPONS CENTER
FKR1A	AIR STATIONS R & D
FKR1B	AIR REWORK FACILITY
FKR3A	AIR ENGINEERING CENTER
FKR3H	AIR PROPULSION CENTER
FKR4B	MISSILE RANGE FACILITY
FKR5	AVIONICS CENTER
FKR7E	AVIATION LOGISTICS CENTER
FR3	AIR STATIONS
FR4	AIR FACILITIES
FR15	SUPPACT
FT1	CHIEF NAVAL ENGR. TRAINING
FT2	CHIEF NAVAL AIR TRAINING
FT6	AIR STATIONS CNET
FT13	AIR TECH TRAINING CEN
FT10	CONSTR, BATTALION UNIT CBC
FT19	ADMINCOM (San Diego only)
FT22	FLECOMBATRACEN (Va. Beach only)
FT28	EDUCATION & TRAINING CEN
FT31	TRAINING CENTER
FT37	SCHOOL CEC OFFICERS
FT55	SCHOOL SUPPLY CORPS
FT78	ED & TRNG PGM DEV CEN
V2	MARCORPS BARRACKS SUPPORT
V3	MC AIR BASE COMMAND
V5	MCAS (less Beaufort)
V5	MCAS (Beaufort only,5 cys)
V8	MC RECRUIT DEPOT
V12	MC DEVELOP.& EDUCATION CEN
V14	MC HEADQUARTERS BATTALION
V15	MC DISTRICT
V17	MC CAMP
V23	MC LOGISTICS BASE
V25	MC AIR GROUND COMBAT CEN

<u>REMAINDER TO</u>: RECEIVING OFFICER
Naval Publications and Forms Center
5801 Tabor Avenue, Phila., PA 19120
FOR IMMEDIATE STOCK

Information on how to obtain additional copies is contained in References.

RECORD OF DOCUMENT CHANGES

Instructions: DISCARD EXISTING SHEET AND INSERT THIS NEW RECORD OF DOCUMENT CHANGES.

This is an inventory of all changes made to this design manual. Each change is consecutively numbered, and each change page in the design manual includes the date of the change which issued it.

Change Number	Description of Change	Date of Change	Page Change
1	Added new cover with revalidation date.	September 1986	Cover
	Added Record of Document Changes.		-
	New Abstract.		iii
	Added to Foreword instruction for sending recommended changes to changed signature to RADM Jones.		v
	Added listing of DM-7 series.		vi
	Deleted Preface.		vii
	Deleted list of Design Manuals.		ix
	New Table of Contents.		vii-xiii
	New Acknowledgments.		xiv
	Added NAVFAC P-418 to Reference list.		7.2-36
	Updated Related Criteria listing.		7.2-37
	Changed Note 1 to Table 4 to reflect Modified Proctor test rather than Standard Proctor.		7.2-47
	Corrected spelling "moisture".		7.2-50
	Corrected equation for Borrow Volume "V+B,		7.2-53
	Added NAVFAC DM's to Reference list.		7.2-57
	Updated Related Criteria listing.		7.2-59
	Added "Figure 3" to first note of Figure 2.		7.2-62
	Deleted the -40 deg., +25 deg., and +30 deg. lines from Figure 3.		7.2-64

RECORD OF DOCUMENT CHANGES (continued)

Change Number	Description of Change	Date of Change	Page Change
	Added NAVFAC D14's to Reference list.		7.2-127
	Changed DM-2 to EM-2.02 in paragraph Related Criteria.		7.2-129
	Changed "Figure 1" to "Figure 6" in first step.		7.2-151
	Added NAVFAC DM's to Reference list.		7.2-175
	Updated Related Criteria listing.		7.2-177
	Added NAVFAC DM's to Reference list.		7.2-244
	Added DD Form 1426.		-

ABSTRACT

This manual covers the application of basic engineering principles of soil mechanics in the design of foundations and earth structures for naval shore facilities. It is intended for use by experienced engineers. The contents include: excavations; compaction, earthwork, and hydraulic fills; analysis of walls and retaining structures; shallow foundations; and deep foundations.

PAGE iv INTENTIONALLY BLANK

FOREWORD

This design manual is one of a series developed from an evaluation of facilities in the shore establishment, from surveys of the availability of new materials and construction methods, and from selection of the best design practices of the Naval Facilities Engineering Command (NAFACENGCOM), other Government agencies, and the private sector. This manual uses, to the maximum extent feasible, national professional society, association, and institute standards in accordance with NAVFACENGCOM policy. Deviations from these criteria should not be made without prior approval of NAVFACENGCOM Headquarters (Code 04).

Design cannot remain static any more than the naval functions it serves or the technologies it uses. Accordingly, recommendations for improvement are encouraged from within the Navy and from the private sector and should be furnished to Commander, Naval Facilities Engineering command (Code 04B), 200 Stovall Street, Alexandria, VA 22332-2300.

This publication is certified as an official publication of the Naval Facilities Engineering Command and has been reviewed and approved in accordance with SECNNAVINST 5600.16, Procedures Governing Review of the Department of the Navy (DN) Publications.

J. P. JONES, JR.
Rear Admiral, CEC, U. S. Navy
Commander
Naval Facilities Engineering Command

SOILS AND FOUNDATIONS DESIGN MANUALS

DM Number	Title
7.01	Soil Mechanics
7.02	Foundations and Earth Structures
7.03	Soil Dynamics, Deep Stabilization, and Special Geotechnical Construction

CONTENTS

Page

CHAPTER 1. EXCAVATIONS

CHAPTER 2. COMPACTION, EARTHWORK, AND HYDRAULIC FILLS

CHAPTER 3. ANALYSIS OF WALLS AND RETAINING STRUCTURES

CHAPTER 4. SHALLOW FOUNDATIONS

CHAPTER 5. DEEP FOUNDATIONS

FIGURES

Figure Title Page

CHAPTER 3 (continued)

CHAPTER 4

Figure Title Page

CHAPTER 4 (continued)

CHAPTER 5

TABLES

Table	Title	Page

CHAPTER 5 (continued)

ACKNOWLEDGEMENTS

Figure	Acknowledgment
Figure 13, Chapter 1	Mazurkiewicz, D.K., Design and Construction of Dry Docks, Trans Tech Publications, Rockport, MA., 1980.
Figure 1, Chapter 2	Sherard, J.L., Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams, Technical Memorandum 645, U.S. Department of the Interior, Bureau of Reclamation.
Figures 5, 6 & 7, Chapter 3	Caquot, A., and Kerisel, J., Tables for the Calculation of Passive Pressure, Active Pressure and Bearing Capacity of Foundations, Gauthier-Villars, Paris.
Figure 16 & 17 Chapter 3	Terzaghi, K. and Peck, R.B., Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, NY.
Figures 23, 24 & 25, Chapter 3	U.S. Steel, Sheet Piling Design Manual, July, 1975.
Figure 36, Chapter 3	Portland Cement Association, Concrete Crib Retaining Walls, Concrete Information No. St. 46, Chicago, IL., May, 1952.
Figures 10 & 11, Chapter 4	Hetenyi, M., Beams on Elastic Foundation, The University of Michigan Press, Ann Arbor, MI.
Figure 14, Chapter 4	Parcher, J.V., and Means, R.E., Soil Mechanics and Foundations, Charles E. Merril Publishing Company, Columbus, OH., 1968.
Figure 2, Chapter 5 (upper panel, right)	Skempton, A.W., The Bearing Capacity of Clays, Proceedings, Building Research Congress, London, 1951.

CHAPTER 1. EXCAVATIONS

Section 1. INTRODUCTION

1. SCOPE. This chapter covers the methods of evaluating the stability of shallow and deep excavations. There are two basic types of excavations: (a) "open excavations" where stability is achieved by providing stable side slopes, and (b) "braced excavations" where vertical or sloped sides are maintained with protective structural systems that can be restrained laterally by internal or external structural elements. Guidance on performance monitoring is given in DM-7.1, Chapter 2.

2. METHODOLOGY. In selecting and designing the excavation system, the primary controlling factors will include: (a) soil type and soil strength parameters; (b) groundwater conditions; (c) slope protection; (d) side and bottom stability; and (e) vertical and lateral movements of adjacent areas, and effects on existing structures.

3. RELATED CRITERIA. For additional criteria on excavations, see the following source:

Subject	Source
Dewatering and Groundwater Control of Deep Excavations....	NAVFAC P-418

Section 2. OPEN CUTS

1. SLOPED CUTS.

a. General. The depth and slope of an excavation, and groundwater conditions control the overall stability and movements of open excavations. In granular soils, instability usually does not extend significantly below the excavation provided seepage forces are controlled. In rock, stability is controlled by depths and slopes of excavation, particular joint patterns, in situ stresses, and groundwater conditions. In cohesive soils, instability typically involves side slopes but may also include materials well below the base of the excavation. Instability below the base of excavation, often referred to as bottom heave, is affected by soil type and strength, depth of cut, side slope and/or berm geometry, groundwater conditions, and construction procedures. Methods for controlling bottom heave are given in DM-7.1, Chapter 6.

b. Evaluation. Methods described in DM-7.1, Chapter 7 may be used to evaluate the stability of open excavations in soils where behavior of such soils can be reasonably determined by field investigation, laboratory testing, and analysis. In certain geologic formations (stiff clays, shales, sensitive clays, clay tills, etc.) stability is controlled by construction procedures, side effects during and after excavation, and inherent geologic planes of weaknesses- Table 1 (modified from Reference 1, Effects of construction on Geotechnical Engineering, by Clough and Davidson) presents a summary of the primary factors controlling excavation slopes in some problem soils. Table 2 (modified from Reference 1 and Reference 2, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures, Departments of Army and Air Force) summarizes measures that can be used for excavation protection for both conventional and problem soils.

2. VERTICAL CUTS. Many cuts in clays will stand with vertical slopes for a period of time before failure occurs. However, changes in the shear strength of the clay with time and stress release resulting from the excavation can lead to progressive deterioration in stability. This process can be rapid in stiff, highly fissured clays, but relatively slow in softer clays. (See DM-7.1, Chapter 7 for critical heights for vertical cuts in cohesive soils.) For cuts in hard unweathered rock, stability is mostly controlled by strength along bedding planes, groundwater condition, and other factors (see DM-7.1, Chapter 6 and Reference 3, Stability of Steep Slopes on Hard Unweathered Rock, by Terzaghi for detailed discussion on the effects of rock discontinuities). Cuts in rock can stand vertical without bolting or anchoring depending on rock quality and joint pattern.

Section 3. TRENCHING

1. SITE EXPLORATION. Individual trenching projects frequently extend over long distances. An exploration program should be performed to define the soil and groundwater conditions over the full extent of the project, so that the design of the shoring system can be adjusted to satisfy the varying site conditions.

2. TRENCH STABILITY. Principal factors influencing trench stability are the lateral earth pressures on the wall support system, bottom heave, and the pressure and erosive effects of infiltrating groundwater (see Chapter 3 and DM-7.1, Chapter 6). External factors which influence trench stability include:

a. Surface Surcharge. The application of any additional load between the edge of the excavation and the intersection of the ground surface with the possible failure plane must be considered in the stability analyses for the excavation.

b. Vibration Loads. The effects of vibrating machinery, blasting or other dynamic loads in the vicinity of the excavation must be considered. The effects of vibrations are cumulative over periods of time and can be particularly dangerous in brittle materials such as clayey sand or gravel.

c. Ground Water Seepage. Improperly dewatered trenches in granular soils can result in quick conditions and a complete loss of soil strength or bottom heave. (See DM-7.1, Chapter 6.)

d. Surface Water Flow. This can result in increased loads on the wall support system and reduction of the shear strength of the soil. Site drainage should be designed to divert water away from trenches.

TABLE 1

Factors Controlling Stability of Sloped Cut in Some Problem Soils

SOIL TYPE	PRIMARY CONSIDERATIONS FOR SLOPE DESIGN
Stiff-fissured Clays and Shales	Field shear resistance may be less than suggested by laboratory tests. Slope failures may occur progressively and shear strengths reduced to residual values compatible with relatively large deformations. Some case histories suggest that the long-term performance is controlled by the residual friction angle which for some shales may be as low as 12 deg. The most reliable design procedure would involve the use of local experience and recorded observations.
Loess and Other Collapsible Soils	Strong potential for collapse and erosion of relatively dry material upon wetting. Slopes in loess are frequently more stable when cut vertical to prevent infiltration. Benches at intervals can be used to reduce effective slope angles. Evaluate potential for collapse as described in DM 7.1, Chapter 1. (See DM-7.3, Chapter 3 for further guidance.)
Residual Soils	Significant local variations in properties can be expected depending on the weathering profile from parent rock. Guidance based on recorded observation provides prudent basis for design.
Sensitive Clays	Considerable loss of strength upon remolding generated by natural or man-made disturbance. Use analyses based on unconsolidated undrained tests or field vane tests.
Talus	Talus is characterized by loose aggregation of rock that accumulates at the foot of rock cliffs. Stable slopes are commonly between 1-1/4 to 1-3/4 horizontal to 1 vertical. Instability is associated with abundance of water, mostly when snow is melting.
Loose Sands	May settle under blasting vibration, or liquify, settle, and lose strength if saturated. Also prone to erosion and piping.

TABLE 2
Factors Controlling Excavation Stability

Construction Activity	Objectives	Comments
Dewatering	To prevent boiling, softening, or heave in excavation bottom, reduce lateral pressures on sheeting, reduce seepage pressures on face of open cut, eliminate piping of fines through sheeting.	Investigate soil compressibility and effect of dewatering on settlement of nearby structures; consider recharging or slurry wall cutoff. Examine for presence of lower aquifer and need to dewater. Install piezometer if needed. Consider effects of dewatering in cavity-laden limestone. Dewater in advance of excavation.
Excavation and Grading	Pipe trenching, basement excavation, site grading.	Analyze safe slopes (see DM-7.1, Chapter 7) or bracing requirement (see Chapter 3), effects of stress reduction on overconsolidated, soft or swelling soils and shales. Consider horizontal and vertical movements in adjacent areas due to excavation and effect on nearby structures. Keep equipment and stockpiles a safe distance from top of excavation.
Excavation Wall Construction	To support vertical excavation walls, to stabilize trenching in limited space.	See Chapter 3 for wall design. Reduce earth movements and bracing stresses, where necessary, by installing lagging on front flange of soldier pile. Consider effect of vibrations due to driving sheet piles or soldier piles. Consider dewatering requirements as well as wall stability in calculating sheeting depth. Movement monitoring may be warranted.

TABLE 2 (continued)
Factors Controlling Excavation Stability

Construction Activity	Objectives	Comments
Blasting	To remove or to facilitate the removal of rock in the excavation.	Consider effect of vibrations on settlement or damage to adjacent areas. Design and monitor or require the contractor to design and monitor blasting in critical areas; require a pre-construction survey of nearby structures.
Anchor or Strut Installation, Wedging of Struts, Pre-stressing Ties	To obtain support system stiffness and interaction.	Major excavations require careful installation and monitoring, e.g., case anchor holes in collapsible soils; measure stress in ties and struts; wedging, etc.

3. SUPPORT SYSTEMS. Excavation support systems commonly used are as follows:

a. Trench Shield. A rigid prefabricated steel unit used in lieu of shoring, which extends from the bottom of the excavation to within a few feet of the top of the cut. Pipes are laid within the shield, which is pulled ahead, as trenching proceeds, as illustrated in Figure 1 (from Reference 4, Cave-In! by Petersen). Typically, this system is useful in loose granular or soft cohesive soils where excavation depth does not exceed 12 feet. Special shields have been used to depths of 30 feet.

b. Trench Timber Shoring. Table 3 illustrates the Occupational Safety and Health Act's minimum requirements for trench shoring. Braces and shoring of trench are carried along with the excavation. Braces and diagonal shores of timber should not be subjected to compressive stresses in excess of:

$$S = 1300 - 20\ L/D$$

where: L = unsupported length (inches)

D = least side of the timber (inches)

S = allowable compressive stress in pounds per square inch of cross section

Maximum Ratio L/D = 50

(1) Skeleton Shoring. Used in soils where cave-ins are expected. Applicable to most soils to depth up to 20 feet. See Figure 2 (from Reference 4) for illustration and guidance for skeleton shoring. Structural components should be designed to safely withstand earth pressures.

(2) Close (Tight) Sheeting. Used in granular or other running soils, compared to skeleton shoring, it is applicable to greater depths. See illustration in Figure 3 (from Reference 4).

(3) Box Shoring. Applicable to trenching in any soil. Depth limited by structural strength and size of timber. Usually limited to 40 feet. See illustration in Figure 4 (from Reference 4).

(4) Telescopic Shoring. Used for excessively deep trenches. See illustration in Figure 5 (Reference 4).

c. Steel Sheeting and Bracing. Steel sheeting and bracing can be used in lieu of timber shoring. Structural members should safely withstand water and lateral earth pressures. Steel sheeting with timber wales and struts have also been used.

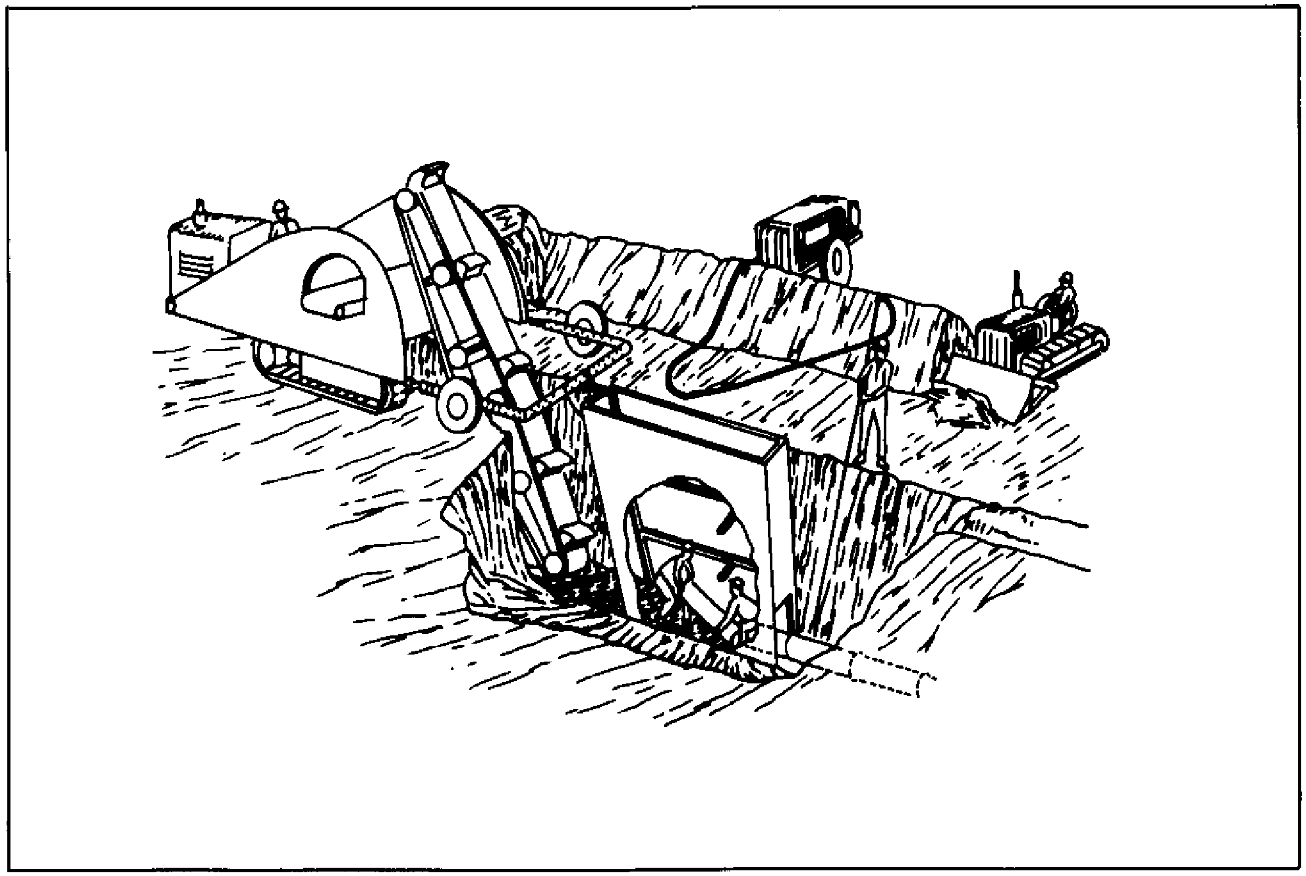


FIGURE 1
Sliding Trench Shield

TABLE 3

OSHA Requirements (Minimum) for Trench Shoring

		Size and Spacing of Members											
		Uprights		Stringers		Cross Braces[1]					Maximum Spacing	
						Width of Trench						
Depth of Trench	Kind or Condition of Earth	Minimum Dimension	Maximum Spacing	Minimum Dimension	Maximum Spacing	Up to 3 feet	4 to 6 feet	7 to 9 feet	10 to 12 feet	13 to 15 feet	Vertical	Hori-zontal
Feet		Inches	Feet	Inches	Feet	Inches	Inches	Inches	Inches	Inches	Feet	Feet
5 to 10	Hard, compact	3x4 or 2x6	6	....	....	2 x 6	4 x 4	4 x 6	6 x 5	6 x 8	4	6
	Likely to crack	3x4 or 2x6	3	4 x 6	4	2 x 6	4 x 4	4 x 6	6 x 6	6 x 8	4	6
	Soft, sandy, or filled	3x4 or 2x6	Close sheeting	4 x 6	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8	4	6
	Hydrostatic pressure	3x4 or 2x6	Close sheeting	6 x 8	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8	4	6
11 to 15	Hard	3x4 or 2x6	4	4 x 6	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8	4	6
	Likely to crack	3x4 or 2x6	2	4 x 6	4	4 x 4	4 x 6	6 x 6	6 x 8	8 x 8		6
	Soft, sandy or filled	3x4 or 2x6	Close sheeting	4 x 6	4	4 x 6	6 x 6	6 x 8	8 x 8	8 x 10	4	6
	Hydrostatic pressure	3x6	Close sheeting	8 x 10	4	4 x 6	6 x 6	6 x 8	8 x 8	8 x 10	4	6

[1] Trench jacks may be used in lieu of, or in combination with, cross braces.
Where desirable, steel sheet piling and bracing of equal strength may be substituted for wood.

TABLE 3 (continued)
OSHA Requirements (Minimum) for Trench Shoring

		Size and Spacing of Members										
		Uprights		Stringers		Cross Braces[1]					Maximum Spacing	
						Width of Trench						
Depth of Trench	Kind or Condition of Earth	Minimum Dimension	Maximum Spacing	Minimum Dimension	Maximum Spacing	Up to 3 feet	4 to 6 feet	7 to 9 feet	10 to 12 feet	13 to 15 feet	Vertical	Hori-zontal
Feet		Inches	Feet	Inches	Feet	Inches	Inches	Inches	Inches	Inches	Feet	Feet
16 to 20	All kinds or conditions	3x6	Close sheeting	4 x 12	4	4 x 12	6 x 8	8 x 8	8 x 10	10 x 10	4	6
Over 20	All kinds or conditions	3x6	Close sheeting	6 x 8	4	4 x 12	8 x 8	8 x 10	10 x 10	10 x 12	4	

[1] Trench jacks may be used in lieu of, or in combination with, cross braces.
Where desirable, steel sheet piling and bracing of equal strength may be substituted for wood.

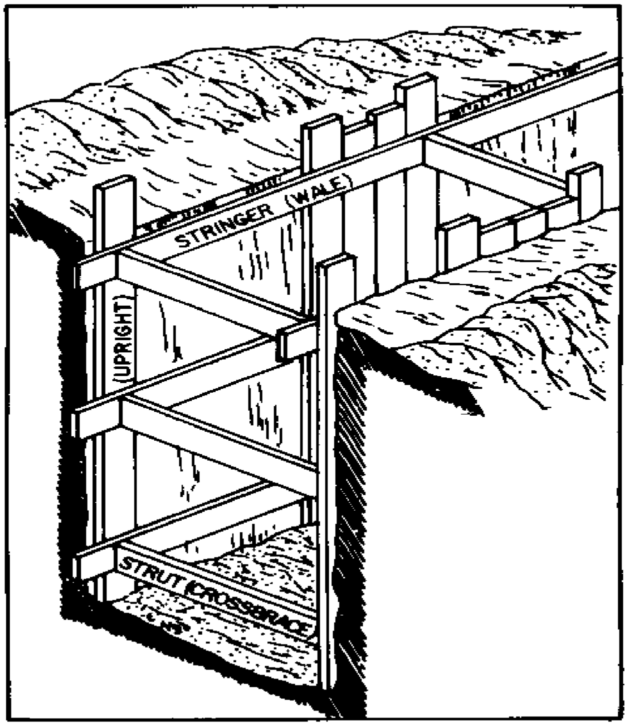


FIGURE 2
Skeleton Shoring

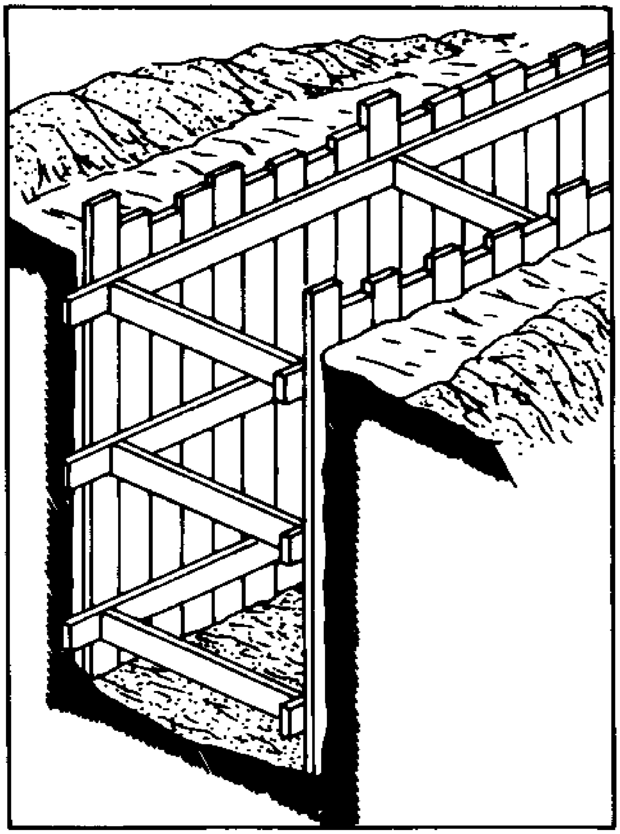


FIGURE 3
Close (Tight) Sheeting

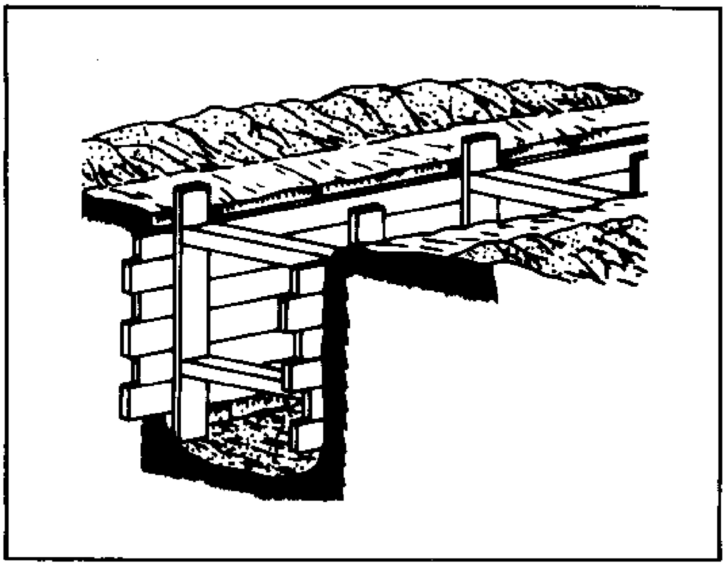


FIGURE 4
Box Shoring

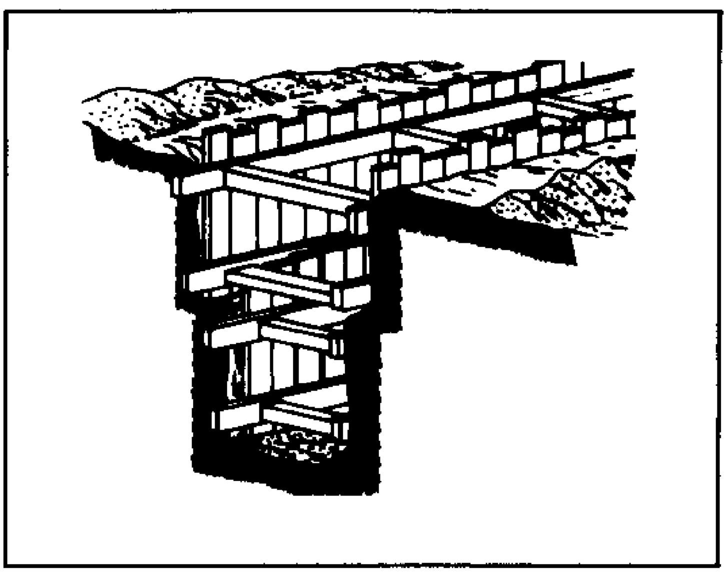


FIGURE 5
Telescopic Shoring

Section 4. BRACED EXCAVATIONS

1. WALL TYPES. Commonly used wall types and limitations to be considered in selection are given in Table 4. Schematics of support systems are shown on Figure 6. A description of wall types listed in Table 4 is presented in Reference 5, Lateral Support Systems and Underpinning, by Goldberg, et al.

2. SELECTION OF SUPPORT SYSTEM. Factors to be considered in selecting types of support systems are given in Table 5.

3. EARTH PRESSURES. The two limiting pressures which may act on the wall are the states of active pressure and passive pressure. Definitions and methods for computing earth pressures are presented in Chapter 3.

For most practical cases, criteria for earth pressures do not exactly conform to the state of active, passive or at rest pressure. Actual earth pressure depends on wall deformation and this in turn depends on several factors. Among the principal factors are: (1) stiffness of wall and support systems; (2) stability of the excavation; and (3) depth of excavation and wall deflection.

The effects of wall deflection on pressure distribution, and differences between strut loads computed from active earth pressure theory and those actually measured for deep excavation in soft clay, are illustrated in Reference 6, Stability of Flexible Structures by Bjerrum, et al. As many different variables affect pressures acting on walls, many types of analyses are available for special situations. (Details concerning these are given in Reference 7, Braced Excavation by Lambe.) Examples of earth pressure computations are given in Chapter 3.

4. OTHER DESIGN AND CONSTRUCTION CONSIDERATIONS. Several factors other than earth pressures affect the selection, design and the performance of braced excavations. See Table 6 for a summary of these factors.

5. LATERAL MOVEMENTS. For well constructed strutted excavations in dense sands and till, maximum lateral wall movements are often less than 0.2% of excavation depth. Lateral movements are usually less for tied back walls. In stiff fissured clays, lateral movements may reach 0.5% or higher depending on quality of construction. In soft clays, a major portion of movement occurs below excavation bottom. Lateral movement may be in the range of 0.5% to 2% of excavation depth, depending on the factor of safety against bottom instability. Higher movements are associated with lesser factors of safety.

6. SOIL SETTLEMENTS BEHIND WALLS. Reference 8, Deep Excavations and Tunneling in Soft Ground by Peck, provides guidance based on empirical observation of settlement behind wall. Settlements up to about 1% of the excavation depth have been measured behind well constructed walls for cuts in sand and in medium stiff clays. In softer clays, this may be as high as 2% and considerably more in very soft clays.

TABLE 4
Types of Walls

Name	Typical EI Values Per Foot (ksf)	Comments
(1) Steel Sheeting	900 - 90,000	- Can be impervious - Easy to handle and construct
(2) Soldier Pile and Lagging	2,000 - 120,000	- Easy to handle and construct - Permits drainage - Can be driven or augered
(3) Cast-in-place or Pre-cast Concrete Slurry Wall (diaphragm walls, see DM-7.3, Chapter 3)	288,000 - 2,300,000	- Can be impervious - Relatively high stiffness - Can be part of permanent structure - Can be prestressed - Relatively less lateral wall movement permitted compared to (1) and (2) - High initial cost - Specialty contractor required to construct - Very large and heavy wall must be used for deep systems - Permits yielding of sub-soils, but precast concrete usually shows less yielding than steel sheeting or soldier pile procedures.
(4) Cylinder Pile Wall	115,000 - 1,000,000	- Secant piles impervious - Relatively high stiffness - Highly specialized equipment not needed for tangent piles - Slurry not needed

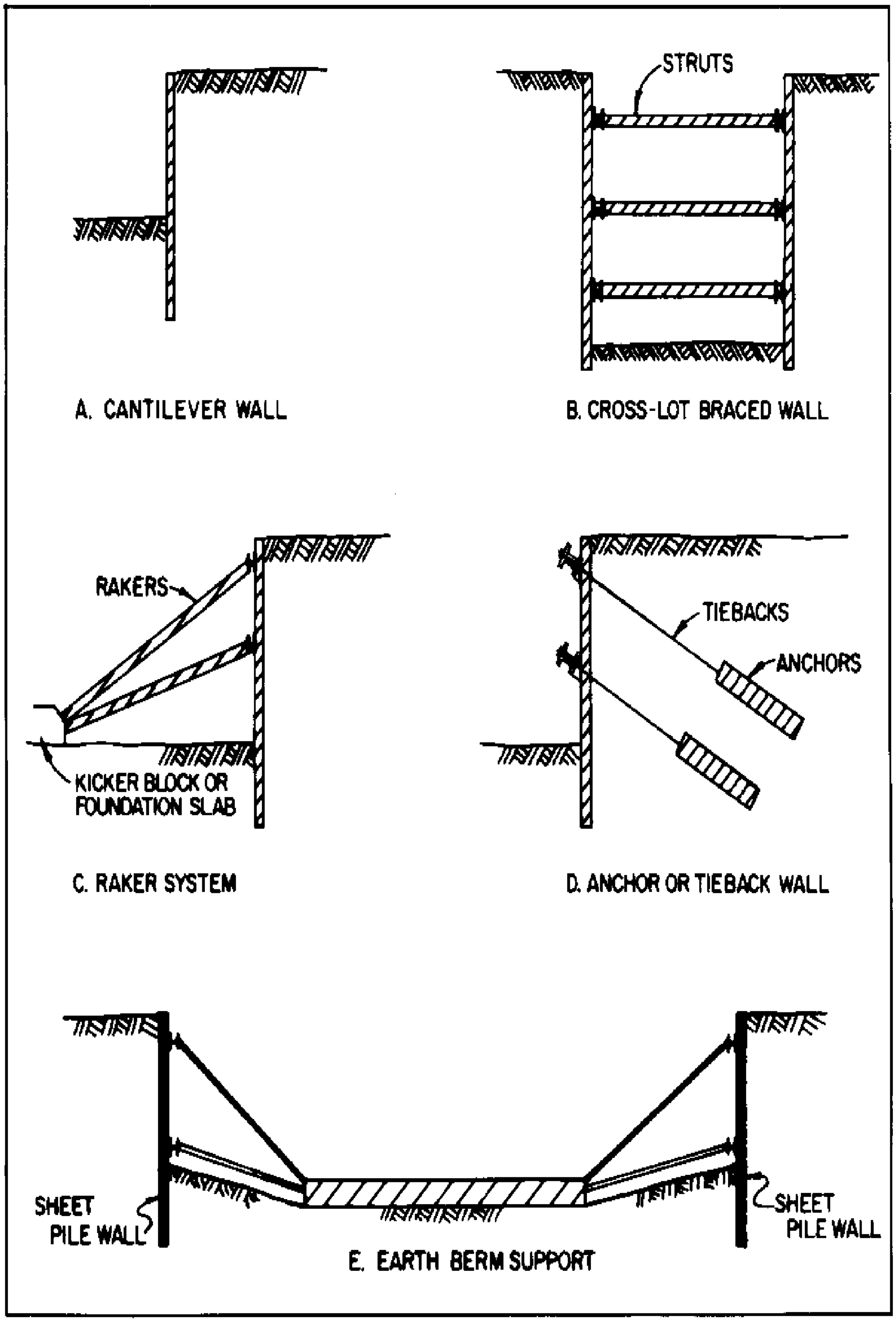


FIGURE 6
Support System - Walled Excavation

TABLE 5
Factors Involved in Choice of A Support System
For A Deep Excavation (> 20 feet)

Requirements	Lends Itself to Use Of	Comments
1. Open excavation area	Tiebacks or rakers or cantilever walls (shallow excavation)	-
2. Low initial cost	Soldier pile or sheetpile walls; combined soil slope with wall	-
3. Use as part of permanent structure	Diaphragm (see DM 7.3 Chapter 3) or cylinder pile walls	Diaphragm wall most common as permanent wall.
4. Deep, soft clay subsurface conditions	Strutted or raker supported diaphragm or cylinder pile walls	Tieback capacity not adequate in soft clays.
5. Dense, gravelly sand or clay subsoils	Soldier pile, diaphragm or cylinder pile	Sheetpiles may lose interlock on hard driving.
6. Deep, overconsolidated clays	Struts, long tiebacks or combination tiebacks and struts.	High in situ lateral stresses are relieved in overconsolidated soils. Lateral movements may be large and extend deep into soil.
7. Avoid dewatering	Diaphragm walls, possibly sheetpile walls in soft subsoils	Soldier pile wall is pervious.
8. Minimize movements	High preloads on stiff strutted or tied-back wall	Analyze for stability of bottom of excavation.
9. Wide excavation (greater than 65 feet wide)	Tiebacks or rakers	Tiebacks preferable except in very soft clay subsoils.
10. Narrow excavation (less than 65 feet wide)	Crosslot struts	Struts more economical but tiebacks still may be preferred to keep excavation open.

TABLE 6
Design Considerations for Braced and Tieback Walls

Design Factor	Comments
1. Water Loads	Often greater than earth load on impervious wall. Recommended piezometers during construction to monitor water levels. Should consider possible lower water pressures as a result of seepage through or under wall. Dewatering can be used to reduce water loads. Seepage under wall reduces passive resistance.
2. Stability	Consider possible instability in any berm or exposed slope. Sliding potential beneath the wall or behind tiebacks should be evaluated. Deep seated bearing failure under weight of supported soil to be checked in weak soils. Stability should consider weight of surcharge or the weight of other facilities in close proximity to excavation.
3. Piping	Loss of ground caused by high groundwater table and silty and fine sand soils. Difficulties occur due to flow beneath wall, through bad joints in walls, or through unsealed sheetpile handling holes. Dewatering may be required.
4. Movements	Movements can be minimized through use of stiff wall supported by preloaded tieback or braced system.
5. Dewatering - recharge	Dewatering reduces loads on wall systems and minimizes possible loss of ground due to piping. May cause settlements and will then need to recharge outside of support system.
6. Surcharge	Construction materials usually stored near wall systems. Allowance should always be made for surcharge.
7. Prestressing of tie backs or struts	Useful to remove slack from system and minimize soil movements.

TABLE 6 (continued)
Design Considerations for Braced and Tieback Walls

<table>
<tr><th>Design Factor</th><th colspan="3">Comments</th></tr>
<tr><td>8. Construction Sequence</td><td colspan="3">The amount of wall movement is dependent on the depth of excavation. The amount of load on the tie backs is dependent on the amount of wall movement which occurs before they are installed. Movements of wall should be checked at every major construction stage. Upper struts should be installed early.</td></tr>
<tr><td>9. Temperature</td><td colspan="3">Struts subject to load fluctuation due to temperature loads; may be important for long struts.</td></tr>
<tr><td>10. Frost Penetration</td><td colspan="3">In very cold climates, frost penetration can cause significant loading on wall system. Design of upper portion of system should be conservative. Anchors may have to be heated. Freezing temperatures also can cause blockage of flow and thus unexpected buildup of water pressure.</td></tr>
<tr><td>11. Earthquakes</td><td colspan="3">Seismic loads may be induced during earthquake. See DM-7.3, Chapter 1.</td></tr>
<tr><td rowspan="9">12. Factors of Safety</td><td rowspan="2">Item</td><td colspan="2">Suggested Minimum Design Factor of Safety for Overall Stability</td></tr>
<tr><td>Permanent</td><td>Temporary</td></tr>
<tr><td>• Earth Berms</td><td>2.0</td><td>1.5</td></tr>
<tr><td>• Cut Slopes</td><td>1.5</td><td>1.3</td></tr>
<tr><td>• Bottom heave above foundation level</td><td>1.5</td><td>1.5</td></tr>
<tr><td>• General stability</td><td>1.5</td><td>1.3</td></tr>
<tr><td>• Bottom heave at foundation level</td><td>2.0</td><td>1.5</td></tr>
<tr><td colspan="3">Note: These values are suggested guidelines only. Design safety factor depends on project requirements.</td></tr>
</table>

7. PROTECTION OF ADJACENT STRUCTURES. Evaluate the effects of braced excavations on adjacent structures to determine whether existing building foundations are to be protected. See DM-7.3, Chapters 2 and 3 on stabilizing foundation soils and methods of underpinning. Figure 7 (modified from Reference 9, <u>Damage to Brick Bearing Wall Structures Caused by Adjacent Braced Cuts and Tunnels</u>, by O'Rourke, et al.) illustrates areas behind a braced wall where underpinning is or may be required.

Factors influencing the type of bracing used and the need for underpinning include:

(a) Lateral distance of existing structure from the braced excavation. Empirical observations on this can be found in Reference 8.

(b) Lowering groundwater can cause soil consolidation and settlement of structures.

(c) Dewatering should be properly controlled to ensure there is no removal of foundation soils outside the excavation.

(d) Tolerance of structures to movement. See DM-7.1, Chapter 5 for evaluation of tolerance of structure to vertical movements. Vertical and lateral movements produce horizontal strains in structure. Guidance on permissible horizontal strains for structures is given in Reference 9.

Section 5. ROCK EXCAVATION

1. OBJECTIVE. Primary objective is to conduct work in such a manner that a stable excavation will be maintained and that rock outside the excavation prism will not be adversely disturbed.

2. PRELIMINARY CONSIDERATIONS. Rock excavation planning must be based on detailed geological data at the site. To the extent possible, structures to be constructed in rock should be oriented favorably with the geological setting. For example, tunnels should be aligned with axis perpendicular to the strike of faults or major fractures. Downslope dip of discontinuities into an open cut should be avoided.

In general, factors that must be considered in planning, designing and constructing a rock excavation are as follows: (1) presence of strike, dip of faults, folds, fractures, and other discontinuities; (2) in situ stresses; (3) groundwater conditions; (4) nature of material filling joints (5) depth and slope of cut; (6) stresses and direction of potential sliding;surfaces; (7) dynamic loading, if any; (8) design life of cut as compared to weathering or deterioration rate of rock face (9) rippability and/or the need for blasting; and (10) effect of excavation and/or blasting on adjacent structures.

The influence of most of these factors on excavations in rock is similar to that of excavations in soil, see DM-7.1, Chapter 7.

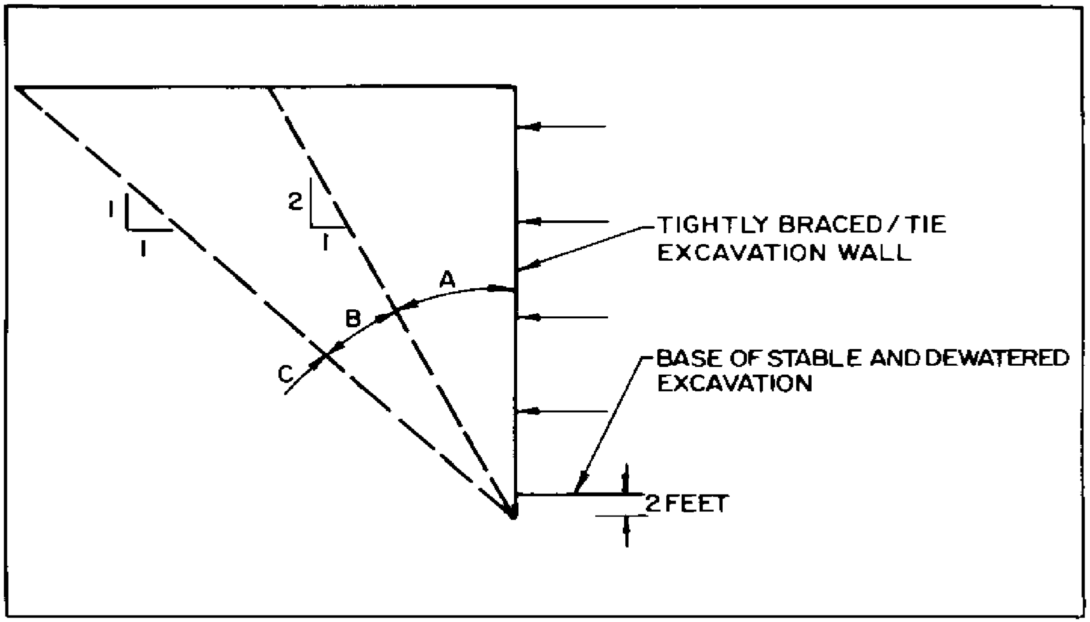


FIGURE 7
General Guidance for Underpinning

3. RIPPABILITY. Excavation ease or rippability can be assessed approximately from field observation in similar materials or by using seismic velocity, fracture spacing, or point load strength index. Figure 8 (from Reference 10, Handbook of Ripping, by Caterpillar Tractor Co.) shows an example of charts for heavy duty ripper performance (ripper mounted on tracked bulldozer) as related to seismic wave velocity. Charts similar to Figure 8 are available from various equipment manufacturers. Figure 8 is for guidance and restricted in applicability to large tractors heavier than 50 tons with engine horsepower greater than 350 Hp. Ripper performance is also related to configuration of ripper teeth, equipment condition and size, and fracture orientation.

Another technique of relating physical properties of rock to excavation ease is shown on Figure 9 (from Reference 11, Logging the Mechanical Character of Rock, by Franklin, et al.) where fracture frequency (or spacing) is plotted against the point load strength index corrected to a reference diameter of 50 mm. (See Reference 12, The Point-Load Strength Test, by Broch and Franklin.)

A third and useful technique is exploration trenching in which the depth of unrippable rock can be established by digging test trenches in rock using rippers (or other excavation equipment) anticipated to be used for the project. The size and shape of the area to be excavated is a significant factor in determining the need for blasting, or the equipment needed to remove the rock.

4. BLASTING. Of major concern is the influence of the blasting on adjacent structures. The maximum particle velocity (the longitudinal velocity of a particle in the direction of the wave that is generated by the blast) is accepted as a criterion for evaluating the potential for structural damage induced by blasting vibration. The critical level of the particle velocity depends on the frequency characteristics of the structure, frequency of ground and rock motion, nature of the overburden, and capability of the structure to withstand dynamic stress. Figure 10 can be used for estimating the maximum particle velocity, which can then be used in Figure 11 (from Reference 13, Blasting Vibrations and Their Effects on Structures, by Bureau of Mines) to estimate potential damage to residential structures. Guidance for human response to blasting vibrations is given in Figure 12 (from Reference 14, Engineering of Rock Blasting on Civil Protects, by Hendron).

Once it has been determined that blasting is required, a pre-blasting survey should be performed. As a minimum, this should include: (a) examination of the site; (b) detailed examination and perhaps photographic records of adjacent structures; and (c) establishment of horizontal and vertical survey control points. In addition, the possibility of vibration monitoring should be considered, and monitoring stations and schedules should be established. During construction, detailed records should be kept of: (a) charge weight, (b) location of blast point and distance from existing structures, (c) delays, and (d) response as indicated by vibration monitoring. For safety, small charges should be used initially to establish a site specific relationship between charge weight, distance, and response.

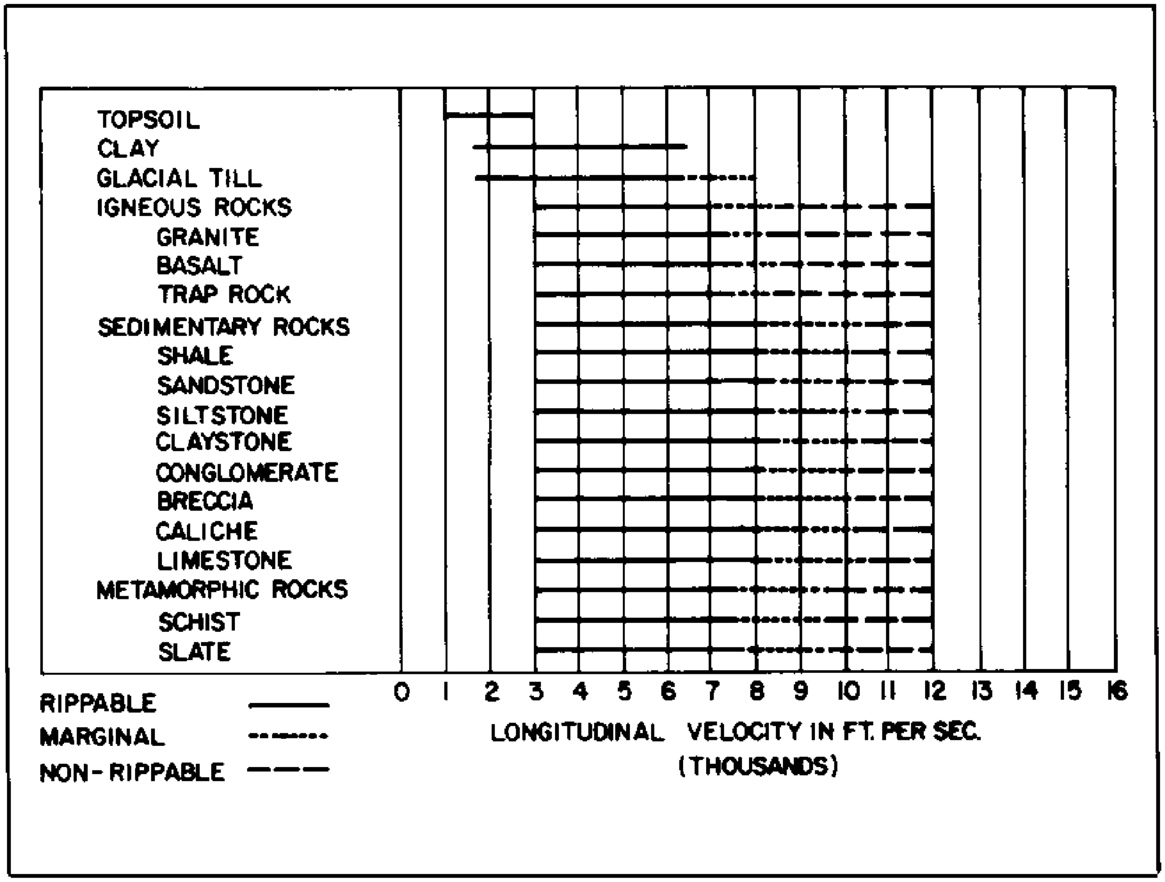
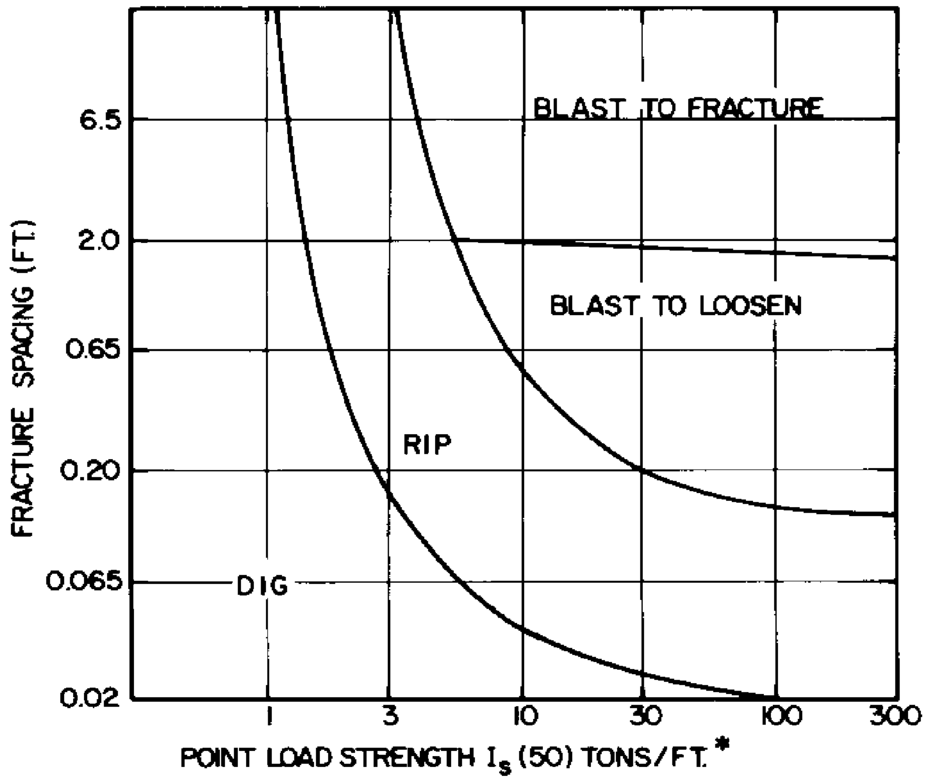
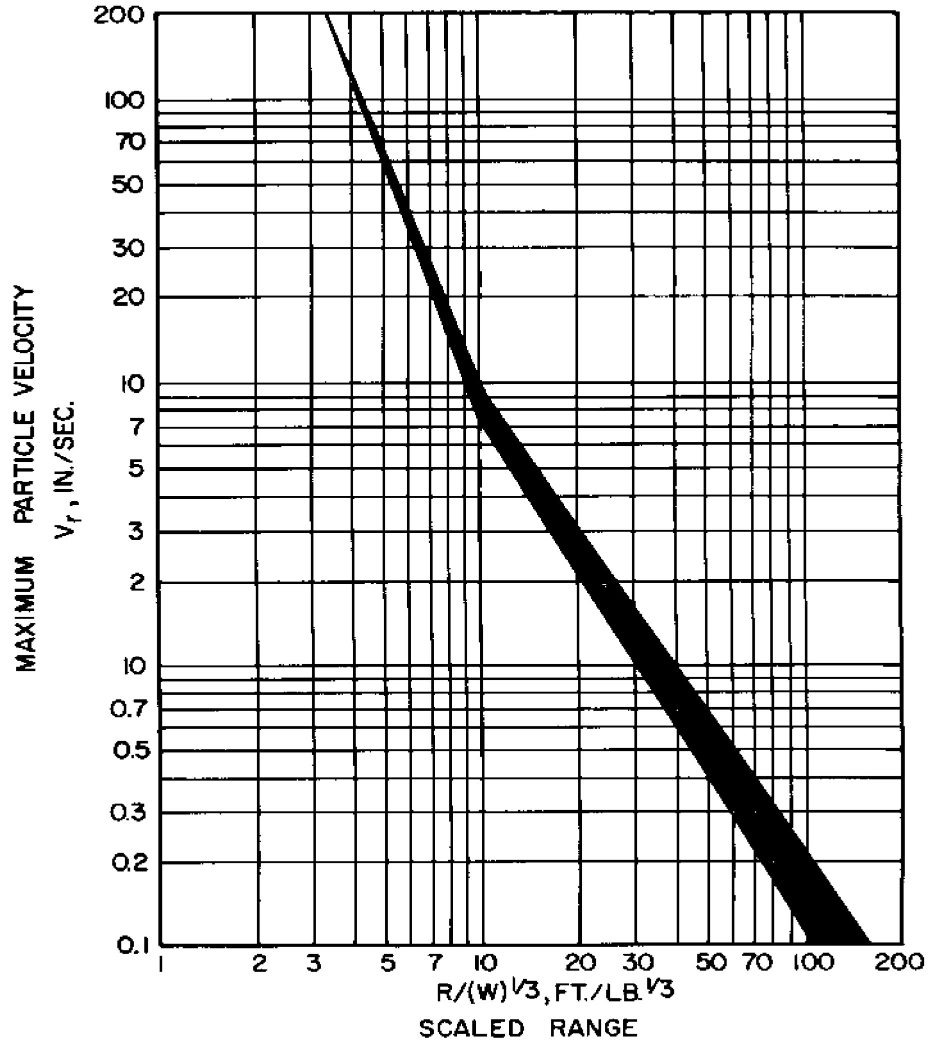


FIGURE 8
Rippability of Subsurface Materials Related to Longitudinal Seismic Velocity for a Heavy Duty Ripper (Tractor-Mounted)



* POINT LOAD STRENGTH CORRECTED TO A REFERENCE DIAMETER OF 50 MM.

FIGURE 9
Suggested Guide for Ease of Excavation



EXAMPLE:

Weight of Explosive Charge = 8 lbs. = W

Distance from Blast Point = 100 ft. = R

$R/(W)^{1/3} = 50$

Peak V_r = 0.5 in/sec from chart

FIGURE 10
Cube Root Scaling Versus Maximum Particle Velocity

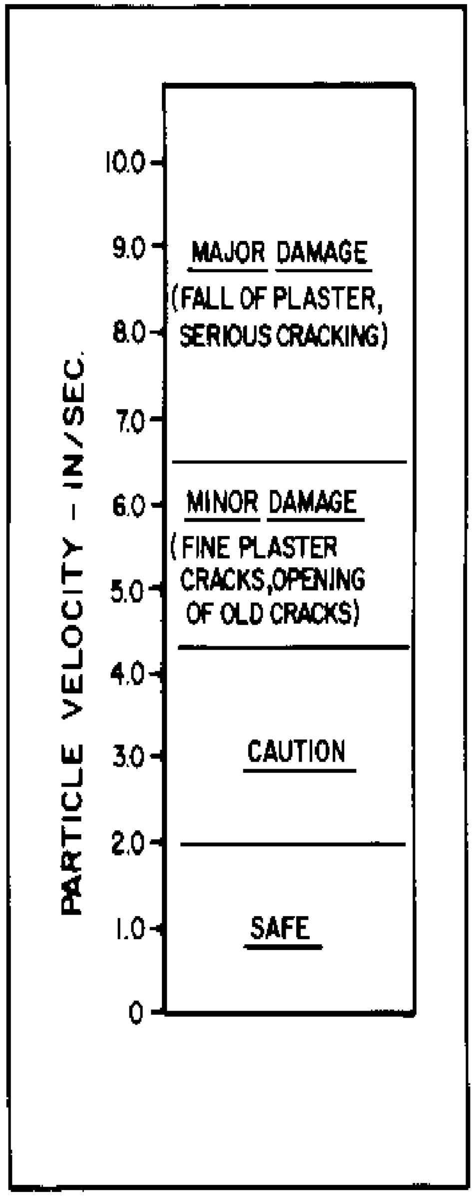


FIGURE 11
Guideline for Assessing Potential for Damage Induced by Blasting Vibration to Residential Structure Founded on Dense Soil or Rock

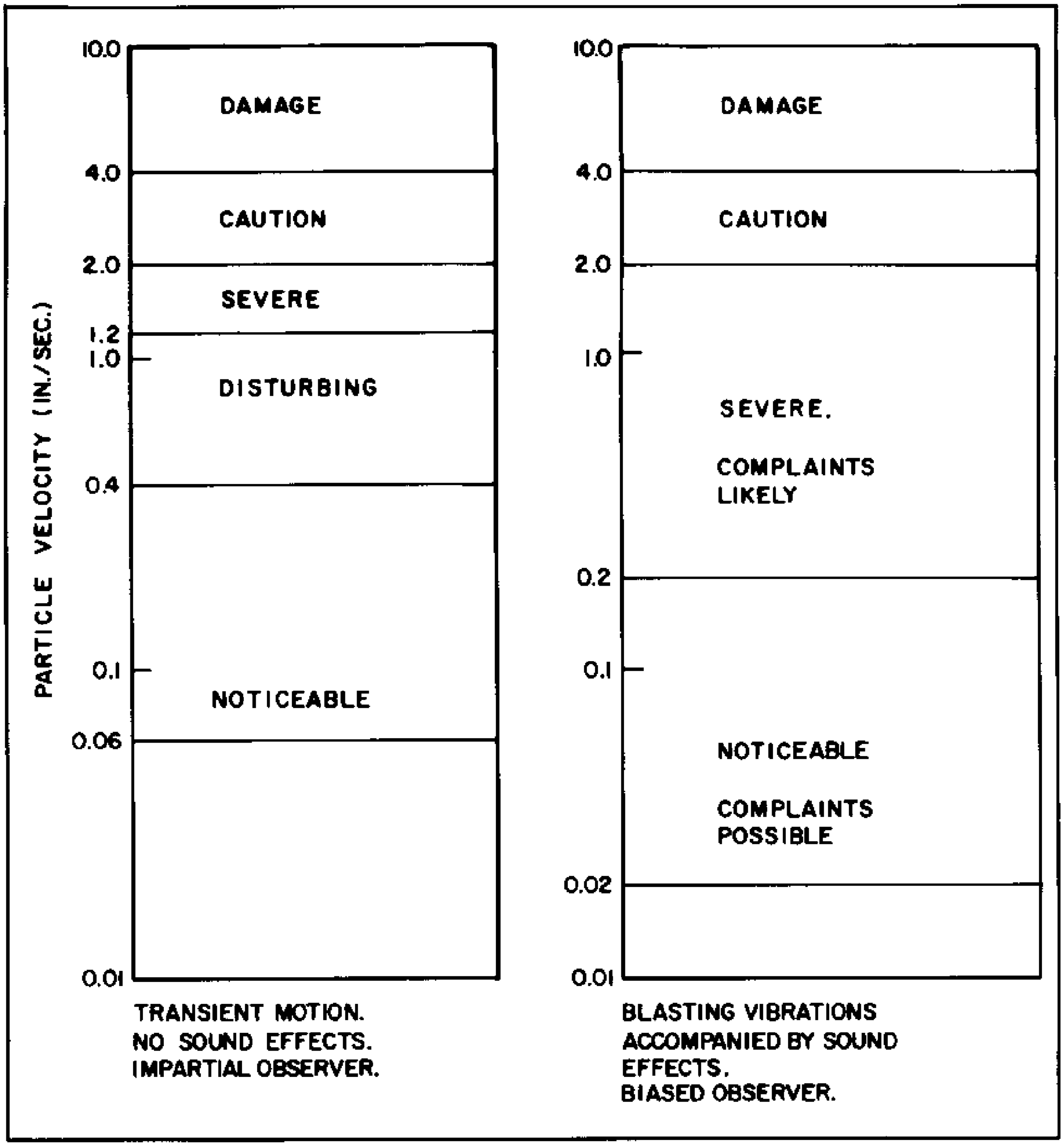


FIGURE 12
Guide for Predicting Human Response to Vibrations and Blasting Effects

Section 6. GROUNDWATER CONTROL

1. APPLICATION. Excavations below the groundwater table require groundwater control to permit construction in the dry and maintain the stability of excavation base and sides. This is accomplished by controlling seepage into the excavation and controlling artesian water pressures below the bottom of the excavation.

2. METHOD. See Table 7 (modified from Reference 15, Control of Groundwater by Water Lowering, by Cashman and Harris) for methods of controlling groundwater, their applicability, and limitations. Wellpoints, deep wells, and sumps are most commonly used. Figures 13(A) (from Reference 2) and 13(B) (from Reference 16, Design and Construction of Dry Docks, by Mazurkiewicz) show a dewatering system using deep wells, and a two stage well point system. Figures 13(C) and 13(D) (from Reference 16) shows details of a wellpoint system, and a deep well with electric submersible pump. See Figure 14 (from Reference 2) for applicable limits of dewatering methods.

3. DESIGN PROCEDURE. See DM-7.1, Chapter 6 for description of design procedures for groundwater control. For additional guidance on groundwater control see NAVFAC P-418.

Section 7. EXCAVATION STABILIZATION, MONITORING, AND SAFETY

1. STABILIZATION. During the planning and design stage, if analyses indicate potential slope instability, means for slope stabilization or retention should be considered. Some methods for consideration are given in Chapter 3.

On occasion, the complexity of a situation may dictate using very specialized stabilization methods. These may include grouting and injection, ground freezing, deep drainage and stabilization, such as vacuum wells or electro-osmosis (see DM-7.3, Chapter 2), and diaphragm walls (see DM-7.3, Chapter 3).

2. MONITORING. During excavation, potential bottom heave, lateral wall or slope movement, and settlement of areas behind the wall or slope should be inspected carefully and monitored if critical. Monitoring can be accomplished by conventional survey techniques, or by more sophisticated means such as heave points, settlement plates, extensometers or inclinometers, and a variety of other devices. See DM-7.1, Chapter 2.

3. SAFETY. Detailed safety requirements vary from project to project. As a guide, safety requirements are specified by OSHA, see Reference 17, Public Law 91-596. A summary of the 1980 requirements follows:

a. OSHA Rules.

(1) Banks more than 4 feet high shall be shored or sloped to the angle of repose where a danger of slides or cave-ins exists as a result of excavation.

TABLE 7

Methods of Groundwater Control

Method	Soils Suitable For Treatment	Uses	Comments
1. Sump pumping	Clean gravels and coarse sands.	Open shallow excavations.	Simplest pumping equipment. Fines easily removed from ground. Encourages instability of formation.
2. Wellpoint systems with suction pumps	Sandy gravels down to fine sands (with proper control can be also used in silty sands).	Open excavations including pipe trench excavations.	Quick and easy to install in suitable soils. Suction lift is limited to about 18 feet. If greater lift is needed multi-stage installation is necessary.
3. Deep wells with electric submersible pumps	Gravels to silty fine sands, and water bearing rocks.	Deep excavations in, through or above water bearing formations.	No limitation on depth of drawdown. Wells can be designed to draw water from several layers throughout its depth. Wells can be sited clear of working area.
4. Jet eductor system using high pressure water to create vacuum as well as to lift the water	Sands, silty sands and sandy silts.	Deep excavations in space so confined that multistage wellpointing cannot be used.	No limitation on depth of drawdown.

TABLE 7 (continued)
Methods of Groundwater Control

Method	Soils Suitable For Treatment	Uses	Comments
5. Sheet piling cut-off	All types of soil (except boulder beds). Tongue and groove wood sheeting utilized for shallow excavations in soft and medium soils.	Practically unrestricted use.	Well-understood method using readily available plant. Rapid installation. Steel can be incorporated in permanent works or recovered. Sump pumping may be required. Estimate seepage flow based on 0.01 gpm/sq ft of wall per foot of differential head. Decrease interlock leakage by filling interlock with sawdust, bentonite, cement grout, or similar materials.
6. Slurry trench cut-off (see DM-7.3, Chapter 3 and DM-7.1, Chapter 6)	Silts, sands, gravels, and cobbles.	Practically unrestricted. Extensive curtain walls around open excavations.	Rapidly installed. Can be keyed into impermeable strata such as clays or soft shales. May be impractical to key into hard or irregular bedrock surfaces, or in open gravels.
7. Freezing (see DM-7.3, Chapter 2)			
a. Ammonium/brine refrigerator	All types of saturated soils and rock.	Formation of ice in the voids stops water.	Treatment effective from working surface outwards. Better for large applications of long duration. Treatment takes longer time to develops.

TABLE 7 (continued)
Methods of Groundwater Control

Method	Soils Suitable For Treatment	Uses	Comments
b. Liquid nitrogen refrigerant	All types of saturated soils and rock.	Formation of ice in the voids stops water.	Better for small applications of short duration where quick freezing is required. Liquid nitrogen is expensive. Requires strict site control. Some ground heave occurs.
8. Diaphragm structural walls			
a. Diaphragm walls (structural concrete) (see DM-7.3, Chapter 3)	All soil types including those containing boulders.	Deep basements, underground construction, shafts.	Can be designed to form part of a permanent foundation. Particularly efficient for circular excavations. Can be keyed into rock. Minimum vibration and noise. Can be used in restricted space. Can be put down very close to existing foundation.
b. Contiguous bored pile walls or impervious wall of mixed in place piles	All soil types but penetration through boulders may be difficult and costly	Deep basements, underground construction, shafts.	A rapidly installed, form of diaphragm wall. Can be keyed into impermeable strata such as clays or soft shales.

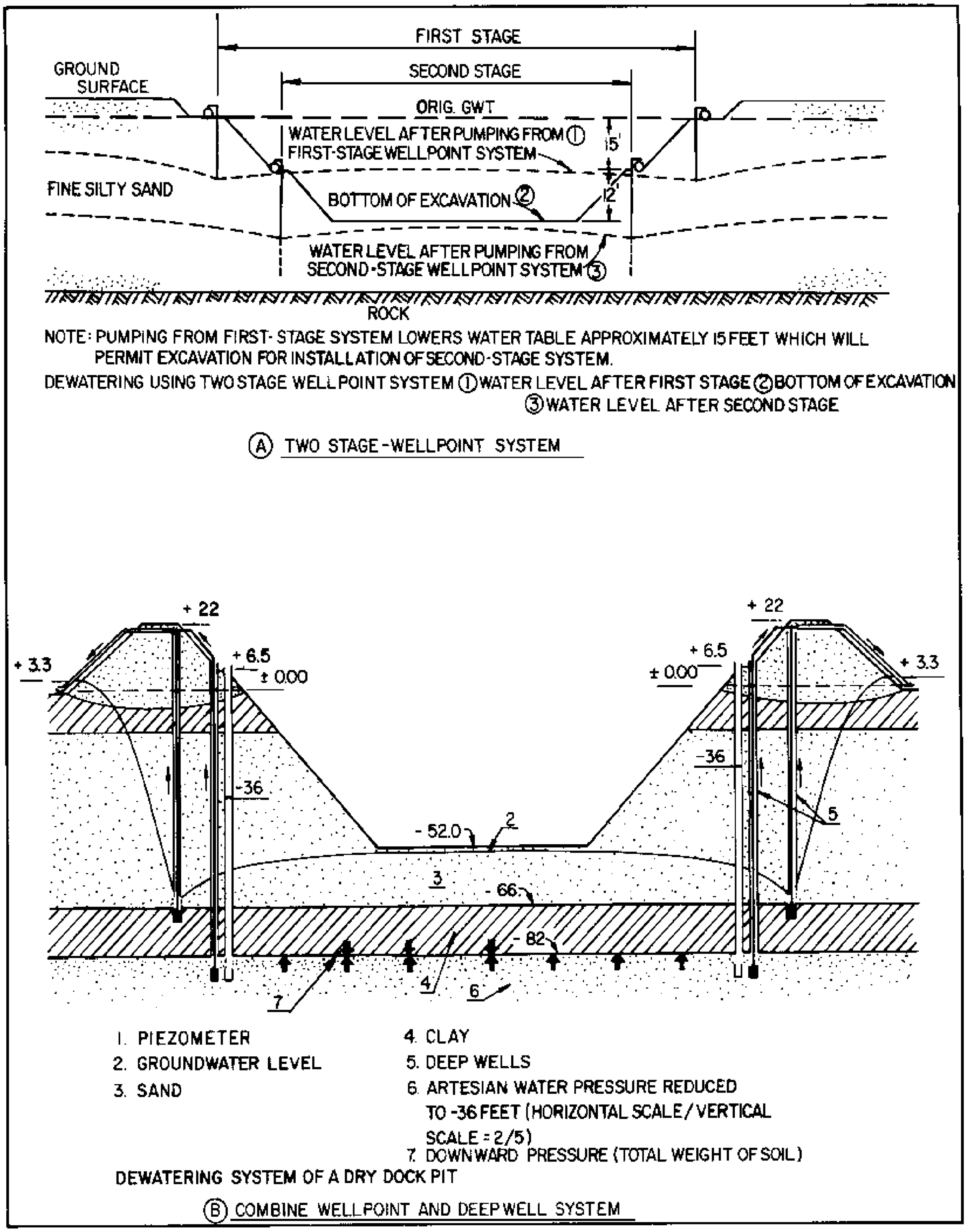


FIGURE 13
Methods of Construction Dewatering

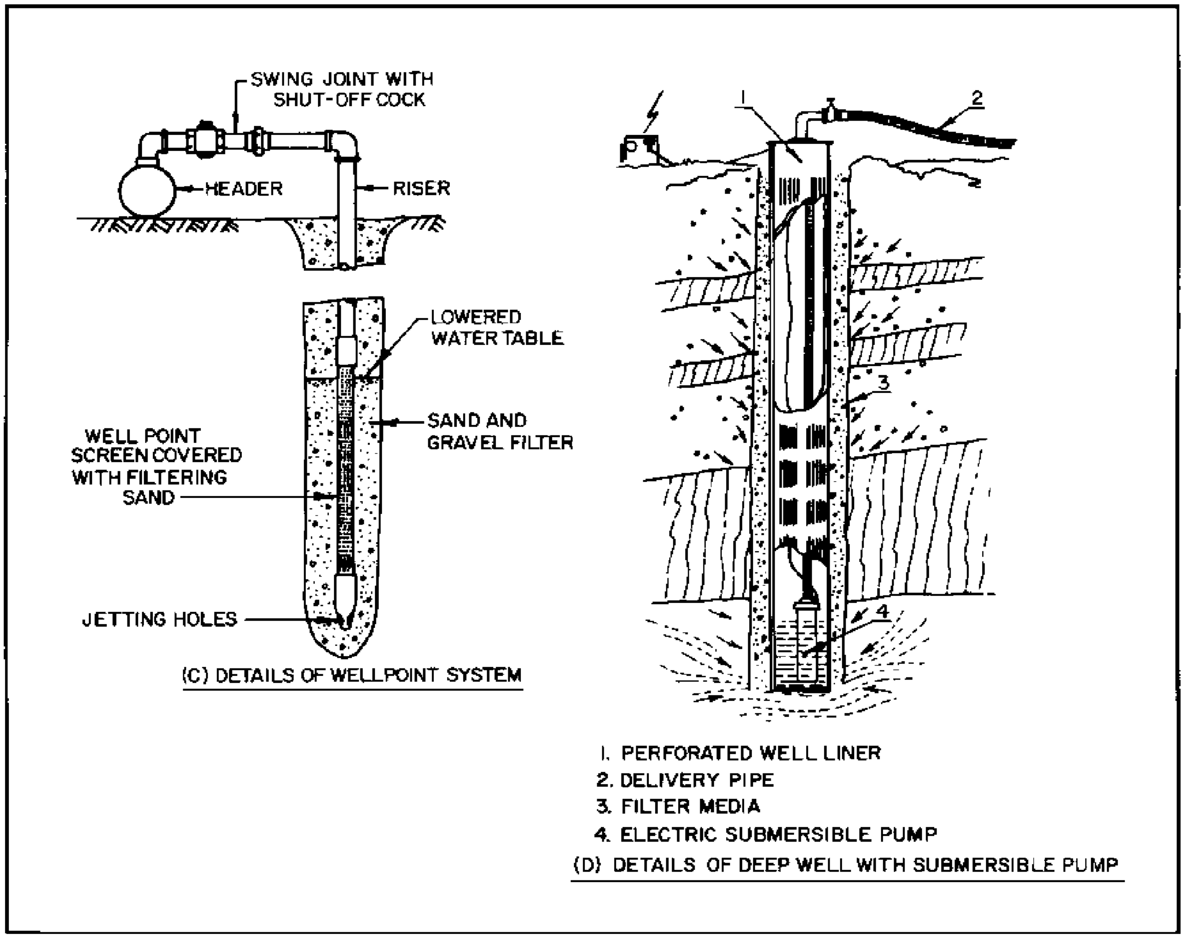


FIGURE 13 (continued)
Methods of Construction Dewatering

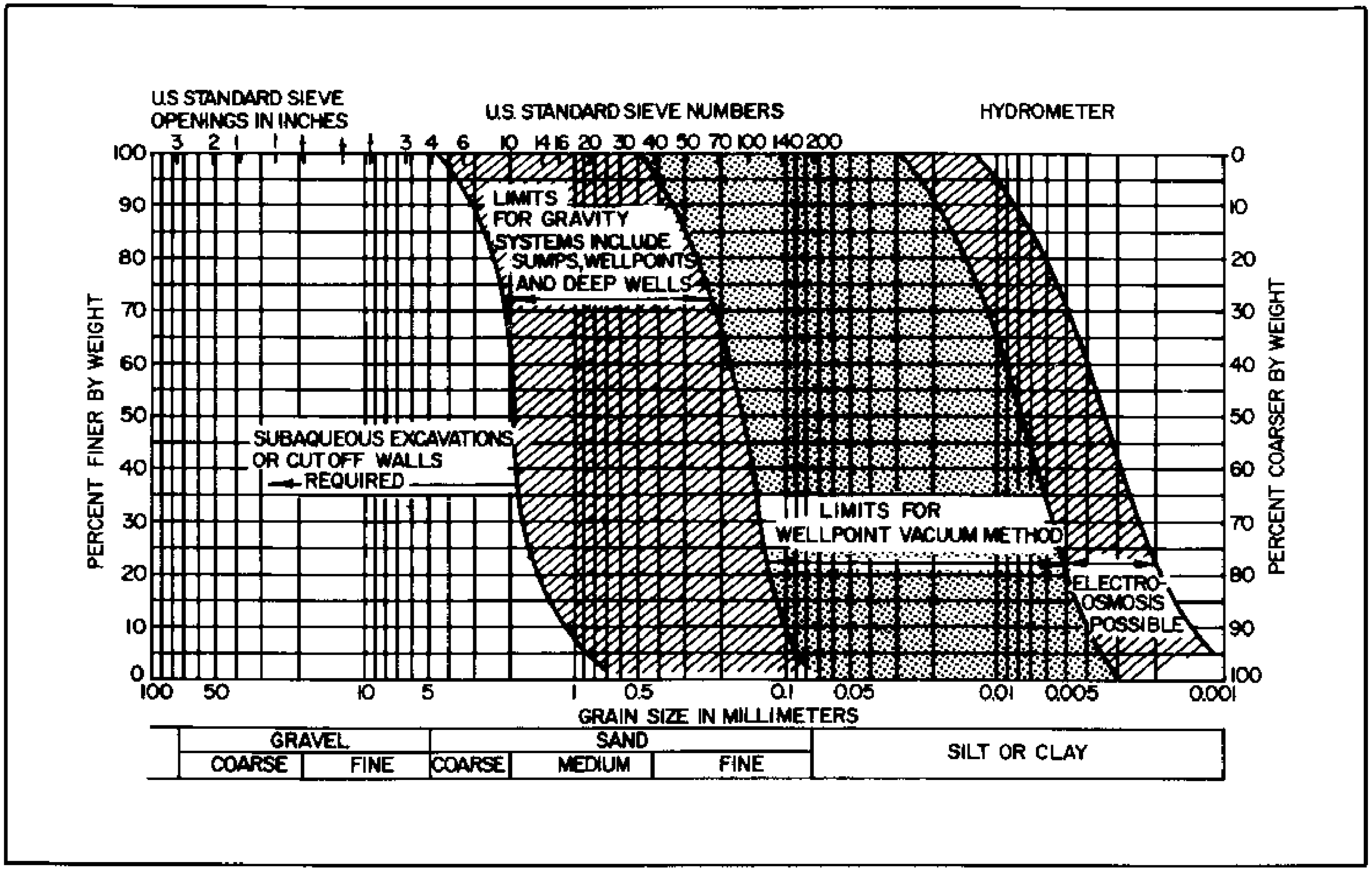


FIGURE 14
Limits of Dewatering Methods Applicable to Different Soils

(2) Sides of trenches in unstable or soft material, 4 feet or more in depth, shall be shored, sheeted, braced, sloped, or otherwise supported by means of sufficient strength to protect the employee working within them.

(3) Sides of trenches in hard or compact soil, including embankments, shall be shored or otherwise supported when the trench is more than 4 feet in depth and 8 feet or more in length. In lieu of shoring, the sides of the trench above the 4-foot level may be sloped to preclude collapse, but shall not be steeper than a 1-foot rise to each 1/2-foot horizontal. When the outside diameter of a pipe is greater than 6 feet, a bench of 4-foot minimum shall be provided at the toe of the sloped portion.

(4) Materials used for sheeting and sheet piling, bracing, shoring, and underpinning shall be in good serviceable condition. Timbers used shall be sound and free from large or loose knots, and shall be designed and installed so as to be effective to the bottom of the excavation.

(5) Additional precautions by way of shoring and bracing shall be taken to prevent slides or cave-ins when (a) excavations or trenches are made in locations adjacent to backfilled excavations; or (b) where excavations are subjected to vibrations from railroad or highway traffic, operation of machinery, or any other source.

(6) Employees entering bell-bottom pier holes shall be protected by the installation of a removable-type casing of sufficient strength to resist shifting of the surrounding earth. Such temporary protection shall be provided for the full depth of that part of each pier hole which is above the bell. A lifeline, suitable for instant rescue and securely fastened to the shafts, shall be provided. This lifeline shall be individually manned and separate from any line used to remove materials excavated from the bell footing.

(7) Minimum requirements for trench timbering shall be in accordance with Table 3.

(8) Where employees are required to be in trenches 3 feet deep or more, ladders shall be provided which extend from the floor of the trench excavation to at least 3 feet above the top of the excavation. They shall be located to provide means of exit without more than 25 feet of lateral travel.

(9) Bracing or shoring of trenches shall be carried along with the excavation.

(10) Cross braces or trench jacks shall be placed in true horizontal position, spaced vertically, and secured to prevent sliding, falling, or kickouts.

(11) Portable trench boxes or sliding trench shields may be used for the protection of employees only. Trench boxes or shields shall be designed, constructed, and maintained to meet acceptable engineering standards.

(12) Backfilling and removal of trench supports shall progress together from the bottom of the trench. Jacks or braces shall be released slowly, and in unstable soil, ropes shall be used to pull out the jacks or braces from above after employees have cleared the trench.

REFERENCES

1. Clough, G.W. and Davidson, R.R., Effects of Construction on Geotechnical Engineering, Specialty Session No. 3, Relationship Between Design and Construction in Soil Engineering, Ninth International Conference on Soil Mechanics and Foundation Engineering, Tokyo, 1977.

2. Departments of the Army and the Air Force, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), TM 5/818-1/AFM 88-3, Chapter 7, Washington, DC, 1979.

3. Terzaghi, K., Stability of Steep Slopes on Hard Unweathered Rock, Norwegian Geotechnical Institute, Publication No. 50, 1963.

4. Petersen, E.V., Cave-In!, Roads and Engineering Construction, November 1963, December 1963, January 1964.

5. Goldberg, D.T., Jaworkski, W.E., and Gordon, M.D., Lateral Support Systems and Underpinning, Vol. II, Design Fundamentals, Vol. III. Construction Methods, Federal Highway Administration, Report Nos. FHWA-RD-75-129, 130, 1976.

6. Bjerrum, L., Clausen, J.F. and Duncan, J.M., Stability of Flexible Structures, General Report, Proceedings, Fifth-International European Conference on Soil Mechanics and Foundation Engineering, Vol. 11, 1977.

7. Lambe, T.W., Braced Excavation, 1970 Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, June 22-24, Cornell University, ASCE, 1971.

8. Peck, R. B., Deep Excavations and Tunneling in Soft Ground, Proceedings, Seventh International Conference on Soil Mechanics and Foundation Engineering, State-of-the-Art Vol. 1, 1969.

9. O'Rourke, T.P., Cording, E.J. and Boscardin, M., Damage to Brick Bearing Wall Structures Caused by Adjacent Braced Cuts and Tunnels, Large Ground Movements, John Wiley & Sons, 1977.

10. Caterpillar Tractor Co., Handbook of Ripping, Fifth Edition, Peoria, IL., 1975.

11. Franklin, J.A., Broch, E. and Walton, G., Logging the Mechanical Character of Rock, Transactions, Institution of Mining and Metallurgy January 1971.

12. Broch, E. and Franklin, J.A., The Point-load Strength Test, International Journal Rock Mechanics and Mining Science, Vol. 9, 1972.

13. Bureau of mines, Blasting Vibrations and Their Effect on Structures, United States Department of the Interior, 1971.

14. Hendron, A.J., Engineering of Rock Blasting on Civil Projects, Rock Excavation Seminar Lectures, ASCE, New York, October, 1976.

15. Cashman, P.M. and Harris, E.T., Control of Groundwater by Water Lowering, Conference on Ground Engineering, Institute of Civil Engineers, London, 1970.

16. Mazurdiewicz, B.K., Design and Construction of Dry Docks, Trans Tech Publications, Rockport, MA, 1980.

17. Public Law 91-596 (Williams-Steiger Act), Occupational Safety and Health Act (OSHA) of 1970, Dec. 29, 1970.

18. Naval Facilities Engineering Command P-Publication, P-418, Dewatering and Groundwater Control. Copies may be obtained from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120.

CHAPTER 2. COMPACTION, EARTHWORK, AND HYDRAULIC FILLS

Section 1. INTRODUCTION

1. SCOPE. This chapter concerns design and construction of compacted fills and performance of compacted materials. Compaction requirements are given for various applications and equipment. Earthwork control procedures end analysis of control test data are discussed. Guidance on hydraulic fills is also included.

2. RELATED CRITERIA. For additional criteria concerned with compaction and earthwork operations, consult the following sources:

Subject	Source
Pavements	NAVFAC DM-5.04
Flexible Pavement Design for Airfield	NAVFAC DM-21.03
Dredging	NAVFAC DM-26.03
Types of Dredging Equipment	NAVFAC DM-38.02

3. PURPOSE OF COMPACTION.

(1) Reduce material compressibility.
(2) Increase material strength.
(3) Reduce permeability.
(4) Control expansion.
(5) Control frost susceptibility.

4. APPLICATIONS. The principal uses of compacted fill include support of structures or pavements, embakments for water retention or for lining reserviors and canals, and backfill surrounding structures or buried utitlities.

5. TYPES OF FILL.

a. Controlled Compacted Fills. Properly placed compacted fill will be more rigid and uniform and have greater strength than most natural soils.

b. Hydraulic Fills. Hydraulic fills cannot be compacted during placement and therefore it is important that the source materials be selected carefully.

c. Uncontrolled Fills. These consist of soils or industrial and domestic wastes, such as ashes, slag, chemical wastes, building rubble, and refuse. Use of ash, slag, and chemical waste is stringently controlled and current Environmental Protection Agency or other appropriate regulations must be considered.

Section 2. EMBANKMENT CROSS-SECTION DESIGN

1. INFLUENCE OF MATERIAL TYPE. Table 1 lists some typical properties of compacted soils which may be used for preliminary analysis. For final analysis engineering property tests are necessary.

a. Utilization. See Table 2 for relative desirability of various soil types in earth fill dams, canals, roadways and foundations. Although practically any nonorganic insoluble soil may be incorporated in an embankment when modern compaction equipment and control standards are employed, the following soils may be difficult to use economically:

(1) Fine-grained soils may have insufficient shear strength or excessive compressibility.

(2) Clays of medium to high plasticity may expand if placed under low confining pressures and/or at low moisture contents. See DM- 7.01, Chapter I for identification of soils susceptible to volume expansion.

(3) Plastic soils with high natural moisture are difficult to process for proper moisture for compaction.

(4) Stratified soils may require extensive mixing of borrow.

2. EMBANKMENTS ON STABLE FOUNDATION. The side slopes of fills not subjected to seepage forces ordinarily vary between 1 on 1-1/2 and 1 on 3. The geometry of the slope and berms are governed by requirements for erosion control and maintenance. See DM-7.01, Chapter 7 for procedures to calculate stability of embankments.

3. EMBANKMENTS ON WEAK FOUNDATIONS. Weak foundation soils may require partial or complete removal, flattening of embankment slopes, or densification. Analyze cross-section stability by methods of DM-7.01, Chapter 7. See DM-7.03, Chapter 2 for methods of deep stabilization, and Chapter 3 for special problem soils.

4. EMBANKMENT SETTLEMENT. Settlement of an embankment is caused by foundation consolidation, consolidation of the embankment material itself, and secondary compression in the embankment after its completion.

a. Foundation Settlement. See DM-7.01, Chapter 5 for procedures to decrease foundation settlement or to accelerate consolidation. See DM-7.03, Chapter 1 for guidance on settlement potential under seismic conditions.

b. Embankment Consolidation. Significant excess pore pressures can develop during construction of fills exceeding about 80 feet in height or for lower fills of plastic materials placed wet of optimum moisture. Dissipation of these excess pore pressures after construction results in settlement. For earth dams and other high fills where settlement is critical, construction pore pressures should be monitored by the methods of DM-7.01, Chapter 2.

TABLE 1
Typical Properties of Compacted Soils

Group Symbol	Soil Type	Range of Maximum Dry Unit Weight, pcf	Range of Optimum Moisture, Percent	Typical Value of Compression: At 1.4 tsf (20 psi) — Percent of Original Height	Typical Value of Compression: At 3.6 tsf (50 psi) — Percent of Original Height	Typical Strength Characteristics: Cohesion (as compacted) psf	Typical Strength Characteristics: Cohesion (saturated) psf	Typical Strength Characteristics: ø (Effective Stress Envelope Degrees)	Typical Strength Characteristics: Tan ø	Typical Coefficient of Permeability ft./min.	Range of CBR Values	Range of Subgrade Modulus k lbs/cu in.
GW	Well graded clean gravels, gravel-sand mixtures.	125 - 135	11 - 8	0.3	0.6	0	0	>38	>0.79	5×10^{-2}	40 - 80	300 - 500
GP	Poorly graded clean gravels, gravel-sand mix	115 - 125	14 - 11	0.4	0.9	0	0	>37	>0.74	10^{-1}	30 - 60	250 - 400
GM	Silty gravels, poorly graded gravel-sand-silt.	120 - 135	12 - 8	0.5	1.1	.....	.....	>34	>0.67	$>10^{-6}$	20 - 60	100 - 400
GC	Clayey gravels, poorly graded gravel-sand-clay.	115 - 130	14 - 9	0.7	1.6	.....	.....	>31	>0.60	$>10^{-7}$	20 - 40	100 - 300
SW	Well graded clean sands, gravelly sands.	110 - 130	16 - 9	0.6	1.2	0	0	38	0.79	$>10^{-3}$	20 - 40	200 - 300
SP	Poorly graded clean sands, sand-gravel mix.	100 - 120	21 - 12	0.8	1.4	0	0	37	0.74	$>10^{-3}$	10 - 40	200 - 300
SM	Silty sands, poorly graded sand-silt mix.	110 - 125	16 - 11	0.8	1.6	1050	420	34	0.67	$5 \times >10^{-5}$	10 - 40	100 - 300
SM-SC	Sand-silt clay mix with slightly plastic fines.	110 - 130	15 - 11	0.8	1.4	1050	300	33	0.66	$2 \times >10^{-6}$	5 - 30	100 - 300
SC	Clayey sands, poorly graded sand-clay-mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	$5 \times >10^{-7}$	5 - 20	100 - 300
ML	Inorganic silts and clayey silts.	95 - 120	24 - 12	0.9	1.7	1400	190	32	0.62	$>10^{-5}$	15 or less	100 - 200
ML-CL	Mixture of inorganic silt and clay.	100 - 120	22 - 12	1.0	2.2	1350	460	32	0.62	$5 \times >10^{-7}$	.....	
CL	Inorganic clays of low to medium plasticity.	95 - 120	24 - 12	1.3	2.5	1800	270	28	0.54	$>10^{-7}$	15 or less	50 - 200
OL	Organic silts and silt-clays, low plasticity.	80 - 100	33 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	50 - 100
MH	Inorganic clayey silts, elastic silts.	70 - 95	40 - 24	2.0	3.8	1500	420	25	0.47	$5 \times >10^{-7}$	10 or less	50 - 100
CH	Inorganic clays of high plasticity	75 - 105	36 - 19	2.6	3.9	2150	230	19	0.35	$>10^{-7}$	15 or less	50 - 150
OH	Organic clays and silty clays	65 - 100	45 - 21	.....	.....	.....	.....	.....	.....	.....	5 or less	25 - 100

Notes:

1. All properties are for condition of "Standard Proctor" maximum density, except values of k and CBR which are for "modified Proctor" maximum density.
2. Typical strength characteristics are for effective strength envelopes and are obtained from USBR data.
3. Compression values are for vertical loading with complete lateral confinement.
4. (>) indicates that typical property is greater than the value shown.
   (..) indicates insufficient data available for an estimate.

TABLE 2

Relative Desirability of Soils as Compacted Fill

Group Symbol	Soil Type	RELATIVE DESIRABILITY FOR VARIOUS USES (No. 1 is Considered the Best, No. 14 Least Desirable)									
		Rolled Earth Fill Dams			Canal Sections		Foundations		Roadways		
									Fills		
		Homogeneous Embankment	Core	Shell	Erosion Resistance	Compacted Earth Lining	Seepage Important	Seepage Not Important	Frost Heave Not Possible	Frost Heave Possible	Surfacing
GW	Well graded gravels, gravel-sand mixtures, little or no fines	–	–	1	1	–	–	1	1	1	3
GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines	–	–	2	2	–	–	3	3	3	–
GM	Silty gravels, poorly graded gravel-sand-silt mixtures	2	4	–	4	4	1	4	4	9	5
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures	1	1	–	3	1	2	6	5	5	1
SW	Well-graded sands, gravelly sands, little or no fines	–	–	3 if gravelly	6	–	–	2	2	2	4
SP	Poorly-graded sands, gravelly sands, little or no fines	–	–	4 if gravelly	7 if gravelly	–	–	5	6	4	–
SM	Silty sands, poorly graded sand-silt mixtures	4	5	–	8 if gravelly	5 erosion critical	3	7	6	10	6
SC	Clayey sands, poorly graded sand-clay mixtures	3	2	–	5	2	4	8	7	6	2
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	6	6	–	–	6 erosion critical	6	9	10	11	–
CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	5	3	–	9	3	5	10	9	7	7
OL	Organic Silts and organic silt-clays of low plasticity	8	8	–	–	7 erosion critical	7	11	11	12	–
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	9	9	–	–	–	8	12	12	13	–
CH	Inorganic clays of high plasticity, fat clays	7	7	–	10	8-vol change critical	9	13	13	8	–
OH	Organic clays of medium high plasticity	10	10	–	–	–	10	14	14	14	–

– Not appropriate for this type of use.

c. Secondary Compression. Even for well-compacted embankments, secondary compression and shear strain can cause slight settlements after completion. Normally this is only of significance in high embankments, and can amount to between 0.1 and 0.2 percent of fill height in three to four years or between 0.3 and 0.6 percent in 15 to 20 years. The larger values are for fine-grained plastic soils.

5. EARTH DAM EMBANKMENTS. Evaluate stability at three critical stages; the end of construction stage, steady state seepage stage, and rapid drawdown stage. See DM-7.1, Chapter 7 for pore pressure distribution at these stages. Seismic forces must be included in the evaluation. Requirements for seepage cutoff and stability dictate design of cross section and utilization of borrow materials.

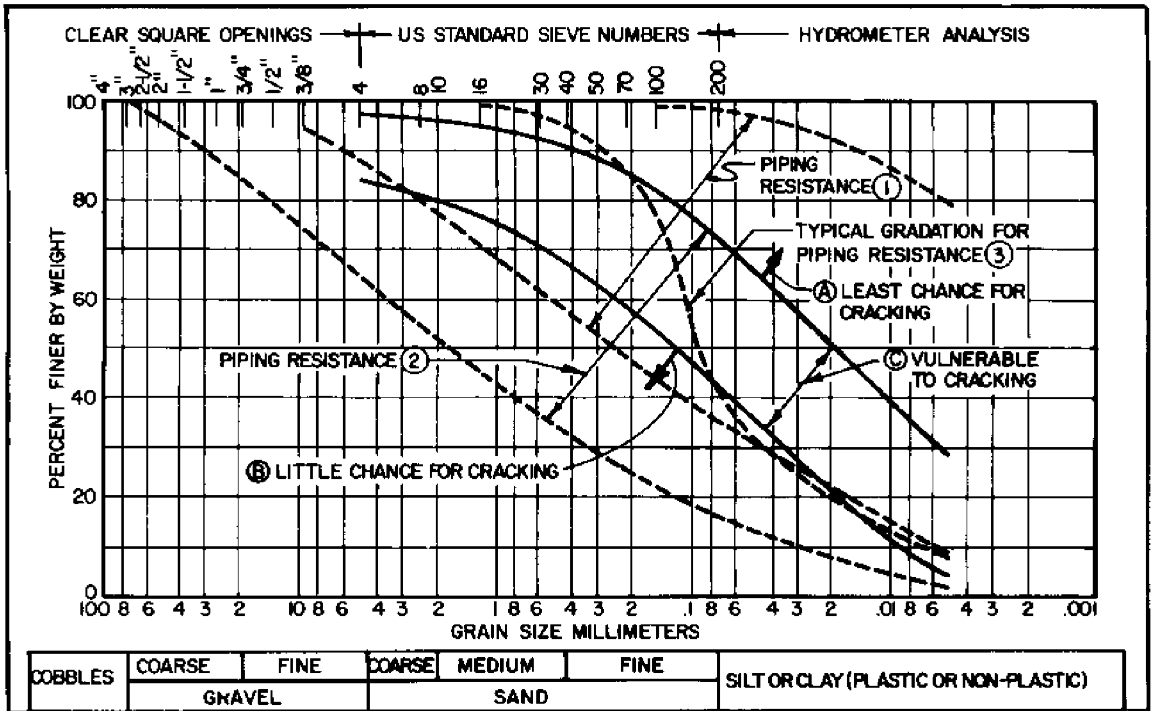
a. Seepage Control. Normally the earthwork of an earth dam is zoned with the least pervious, fine-grained soils in the central zone and coarsest, most stable material in the shell. Analyze seepage by the methods of DM-7.1, Chapter 6.

(1) Cutoff Trench. Consider the practicability of a positive cutoff trench extending to impervious strata beneath the embankment and into the abutments.

(2) Intercepting Seepage. For a properly designed and constructed zoned earth dam, there is little danger from seepage through the embankment. Drainage design generally is dictated by necessity for intercepting seepage through the foundation or abutments. Downstream seepage conditions are more critical for homogeneous fills. See DM-7.1, Chapter 6 for drainage and filter requirements.

b. Piping and Cracking. A great danger to earth dams, particularly those of zoned construction, is the threat of cracking and piping. Serious cracking may result from tension zones caused by differences in stress-strain properties of zoned material. See Figure 1 (Reference 1, Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams, by Sherard) for classification of materials according to resistance to piping or cracking. Analyze the embankment section for potential tension zone development. Place an internal drainage layer immediately downstream of the core to control seepage from possible cracking if foundation settlements are expected to be high.

c. Dispersive soil. Dispersive clays should not be used in dam embankments. Determine the dispersion potential using Table 3 or the method outlined in Reference 2, Pinhole Test for Identifying Dispersive Soils, by Sherard, et al. A hole through a dispersive clay will increase in size as water flows through (due to the breakdown of the soil structure), whereas the size of a hole in a non-dispersive clay would remain essentially constant. Therefore, dams constructed with dispersive clays are extremely susceptible to piping.



CATEGORY	MATERIAL	CHARACTERISTICS
PIPING RESISTANCE:		
①	CL AND CH WITH PI > 15, WELL GRADED SC WITH PI > 15.	GREATEST RESISTANCE TO PIPING, SMALL AND MEDIUM CONCENTRATED LEAKS WILL HEAL THEMSELVES. EMBANKMENT MAY FAIL AS A RESULT OF SLOWLY PROGRESSIVE PIPING CAUSED BY LEAK OF ABOUT 1/2 CFS.
②	CL AND ML WITH PI < 15, WELL GRADED SC AND GC WITH 15 > PI > 7.	INTERMEDIATE RESISTANCE TO PIPING. SAFELY RESISTS SATURATION OF LOWER PORTION OF DOWNSTREAM SLOPE INDEFINITELY. MAY FAIL EVENTUALLY AS A RESULT OF EROSION CAUSED BY A SMALL CONCENTRATED LEAK OR BY PROGRESSIVE SLOUGHING. IF A LARGE LEAK DEVELOPS, PIPING CAUSES FAILURE IN A SHORT TIME.
③	SP AND UNIFORM SM AND ML WITH PI < 7.	LEAST RESISTANCE TO PIPING. USUALLY FAILS IN A FEW YEARS AFTER FIRST RESERVOIR FILLING IF SEEPAGE IS ABLE TO BREAK OUT ON DOWNSTREAM SLOPE. SMALL CONCENTRATED LEAK ON DOWNSTREAM SLOPE CAN CAUSE FAILURE IN A SHORT PERIOD OF TIME. HIGH DENSITY FROM COMPACTION INCREASES RESISTANCE SIGNIFICANTLY.

FIGURE 1
Resistance of Earth Dam Embankment Materials To Piping and Cracking

CATEGORY	MATERIAL	CHARACTERISTICS
CRACKING RESISTANCE		
A	CH WITH $D_{50} < 0.02$ MM AND PI > 20	HIGH POSTCONSTRUCTION SETTLEMENT, PARTICULARLY IF COMPACTED DRY. HAS SUFFICIENT DEFORMABILITY TO UNDERGO LARGE SHEAR STRAINS FROM DIFFERENTIAL SETTLEMENT WITHOUT CRACKING.
B	GC, SC, SM, SP WITH $D_{50} > 0.15$ MM	SMALL POSTCONSTRUCTION SETTLEMENT. LITTLE CHANCE FOR CRACKING UNLESS POORLY COMPACTED AND LARGE SETTLEMENT IS IMPOSED ON EMBANKMENT BY CONSOLIDATION OF THE FOUNDATION.
C	CL, ML AND SM WITH PI < 20, 0.15 MM > D_{50} > 0.02 MM.	MEDIUM TO HIGH POSTCONSTRUCTION SETTLEMENT AND VULNERABLE TO CRACKING. SHOULD BE COMPACTED AS WET AS POSSIBLE CONSISTENT WITH STRENGTH REQUIREMENTS.

FIGURE 1 (continued)
Resistance of Earth Dam Embankment Materials To Piping and Cracking

TABLE 3
Clay Dispersion Potential

*Percent Dispersion	Dispersive Tendency
Over 40	Highly Dispersive (do not use)
15 to 40	Moderately Dispersive
0 to 15	Resistant to Dispersion

*The ratio between the fraction finer than 0.005 mm in a soil-water suspension that has been subjected to a minimum of mechanical agitation, and the total fraction finer than 0.005 mm determined from a regular hydrometer test x 100.

Section 3. COMPACTION REQUIREMENTS AND PROCEDURES

1. COMPACTION REQUIREMENTS.

a. Summary. See Table 4 for a summary of compaction requirements of fills for various purposes. Modify these to meet conditions and materials for specific projects.

b. Specification Provisions. Specify the desired compaction result. State the required density, moisture limits, and maximum lift thickness, allowing the contractor freedom in selection of compaction methods and equipment. Specify special equipment to be used if local experience and available materials so dictate.

2. COMPACTION METHODS AND EQUIPMENT. Table 5 lists commonly used compaction equipment with typical sizes and weights and guidance on use and applicability.

3. INFLUENCE OF MATERIAL TYPE.

a. Soils Insensitive to Compaction Moisture. Coarse-grained, granular well-graded soils with less than 4 percent passing No. 200 sieve (8 percent for soil of uniform gradation) are insensitive to compaction moisture. (These soils have a permeability greater than about 2 x 10. -3- fpm.) Place these materials at the highest practical moisture content, preferably saturated. Vibratory compaction generally is the most effective procedure. In these materials, 70 to 75 percent relative density can be obtained by proper compaction procedures. If this is substantially higher than Standard Proctor maximum density, use relative density for control. Gravel, cobbles and boulders are insensitive to compaction moisture. Compaction with smooth wheel vibrating rollers is the most effective procedure. Use large scale tests, as outlined in Reference 3, Control of Earth Rockfill for Oroville Dam, by Gordon and Miller.

b. Soils Sensitive to Compaction Moisture. Silts and some silty sands have steep moisture-density curves, and field moisture must be controlled Within narrow limits for effective compaction. Clays are sensitive to moisture in that if they are too wet they are difficult to dry to optimum moisture, and if they are dry it is difficult to mix the water in uniformly. Sensitive clays do not respond to compaction because they lose strength upon remolding or manipulation.

c. Effect of Oversize. Oversize refers to particles larger than the maximum size allowed using a given mold (i.e. No. 4 for 4-inch mold, 3/4 inch for 6-inch mold, 2-inch for a 12-inch mold). Large size particles interfere with compaction of the finer soil fraction. For normal embankment compaction the maximum size cobble should not exceed 3 inches or 50 percent of the compacted layer thickness. Where economic borrow sources contain larger sizes, compaction trials should be run before approval.

TABLE 4
Compaction Requirements

Fill Utilized for:	Required Density, Percent of Modified Proctor	Tolerable Range of Moisture About Optimum, Percent	Maximum Permissible Lift Thickness, Compacted in.	Special Requirements
Support of structure	95	-2 to +2	12	Fill should be uniform. Blending or processing of borrow may be required. For plastic clays, investigate expansion under saturation for various compaction moisture and densities at loads equal to those applied by structure, to determine condition to minimize expansion. Clays that show expansive tendencies generally should be compacted at or above optimum moisture to a density consistent with strength and incompressibility required of the fill.
Lining for canal or small reservoir	90	-2 to +2	6	For thick linings, GW-GC, GC, SC are preferable for stability and to resist erosive forces. Single size silty sands with PI less than five generally are not suitable. Remove fragments larger than 6 inches before compaction.
Earth dam greater than 50 ft. high	95	-1 to +2	12(+)	Utilize least pervious materials as central core and coarest materials in outer shells. Core should be free of lenses, pockets, or layers of pervious material and successive lifts well bonded to each other. Amounts of oversize exceeding 1 percent of total material should be removed from the borrow prior to arrival on the embankment.
Earth dam less than 50 ft. high	92	-1 to +3	12(+)	In small dams that lack elaborate zoning, materials that are the most vulnerable to cracking and piping should be compacted to 98 percent density at moisture content from optimum to 3 percent in excess of optimum.
Support of pavements:				Place coarest borrow materials at top of fill. Investigate expansion of plastic clays placed near pavement subgrade to determine compaction moisture and density that will minimize expansion and provide required soaked CBR values.
Highways.....	See NAVFAC DM-5	-2 to +2	8(+)	
Airfields....	See NAVFAC DM-21	-2 to +2	8(+)	
Backfill surrounding structure	90	-2 to +2	8(+)	Where backfill is to be drained, provide pervious coarse-grained soils. For low walls, do not permit heavy rolling compaction equipment to operate closer to the wall than a distance equal to about 2/3 the unbalanced height of fill at any time. For highwalls or walls of special design, evaluate the surcharge produced by heavy compaction equipment by the methods of Chapter 3 and specify safe distances back of the wall for its operations.

TABLE 4 (continued)
Compaction Requirements

Fill Utilized for:	Required Density, Percent of Modified Proctor	Tolerable Range of Moisture About Optimum, Percent	Maximum Permissible Lift Thickness, Compacted in.	Special Requirements
Backfill in pipe or utility trenches	90	-2 to +2	8(+)	Material excavated from trench generally is suitable for backfill if it does not contain organic matter or refuse. If backfill is fine grained, a cradle for the pipe is formed in natural soil and backfill placed by tamping to provide the proper bedding. Where free draining sand and gravel is utilized, the trench bottom may be finished flat and the granular material placed saturated under and around the pipe and compacted by vibration.
Drainage blanket or filter	90	Thoroughly wetted	8	Ordinarily vibratory compaction equipment is utilized. Blending of materials may be required for homogeneity. Segregation must be prevented in placing and compaction. For compaction adjacent to and above drainage pipe, use hand tamping or light travelling vibrators.
Subgrade of excavation for structure	95	-2 to +2	-	For uniform bearing or to break up pockets of frost susceptible material, scarify the upper 8 to 12 in. of the subgrade, dry or moisten as necessary and recompact. Certain materials, such as heavily preconsolidated clays which will not benefit by compaction, or saturated silts and silty fine sands that become quick during compaction, should be blanketed with a working mat of lean concrete or coarse grained material to prevent disturbance or softening. Depending on foundation conditions revealed in exploration, a substantial thickness of loose soils may have to be removed below subgrade and recompacted, or compacted in place by vibration, or pile driving.
Rock fill	-	Thoroughly wetted	2 to 3 ft.	For fill containing sizes no larger than ft., place in layers not exceeding 24 in., thoroughly wetted and compacted by travel or heavy crawler tractors in spreading. Material with sizes up to 2 ft. may be placed in 3 ft lifts. Placing should be such that the maximum size of rock increases toward the outer slopes. Rocks larger than 1 cu yd in volume should be embedded on the slope.

Notes:

1. Density and moisture content refer to "Modified Proctor" test values, (ASTM D 1557)

2. Generally, a fill compacted dry of OMC will have higher strength and a lower compressibility even after saturation.

3. Compaction of "Coarse-grained, granular soil" is not sensitive to moisture content so long as bulking moisture is avoided. Where practicable, they should be placed saturated and compacted by vibratory methods.

TABLE 5
Compaction Equipment and Methods

Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Sheepsfoot Rollers	For fine-grained soils or dirty coarse-grained soils with more than 20 percent passing No. 200 sieve. Not suitable for clean coarse-grained soils. Particularly appropriate for compaction of impervious zone for earth dam or linings where bonding of lifts is important.	6	4 to 6 passes for fine-grained soil. 6 to 8 passes for coarse-grained soil.	Soil Type / Foot Contact Area sq. ft. / Foot Contact Pressures psi: Fine-grained soil PI>30 / 5 to 12 / 250 to 500 Fine-grained soil PI<30 / 7 to 14 / 200 to 400 Coarse-grained soil / 10 to 14 / 150 to 250 Efficient compaction of soils wet of optimum requires less contact pressure than the same soils at lower moisture contents.	For earth dam, highway and airfield work, articulated self propelled rollers are commonly used. For smaller projects, towed 40 to 60 inch drums are used. Foot contact pressure should be regulated so as to avoid shearing the soil on the third or fourth pass.
Rubber Tire Roller	For clean, coarse-grained soils with 4 to 8 percent passing the No. 200 sieve.	10	3 to 5 coverages	Tire inflation pressures of 35 to 130 psi for clean granular material or base course and subgrade compaction. Wheel load 18,000 to 25,000 lbs.	Wide variety of rubber tire compaction equipment is available. For cohesive soils, light-wheel loads, such as provided by wobble-wheel equipment, may be substituted for heavy-wheel load if lift thickness is decreased. For granular soils, large-size tires are desirable to avoid shear and rutting.
Do.......	For fine-grained soils or well graded, dirty coarse-grained soils with more than 8 percent passing the No. 200 sieve.	6 to 8	4 to 6 coverages	Tire inflation pressures in excess of 65 psi, for fine-grained soils of high plasticity. For uniform clean sands or silty fine sands, use large size tires with pressures of 40 to 50 psi.	
Smooth Wheel Rollers	Appropriate for subgrade or base course compaction of well-graded sand-gravel mixtures.	8 to 12	4 coverages	Tandem type rollers for base course or subgrade compaction 10 to 15 ton weight, 300 to 500 lbs per lineal in. of width of rear roller.	3-wheel rollers obtainable in wide range of sizes. 2-wheel tandem rollers are availabe in the range of 1 to 20 ton weight. 3-Axle tandem rollers are generally used in the range of 10 to 20 tons weight. Very heavy rollers are used for proof rolling of subgrade or base course.
Do....	May be used for fine-grained soils other than in earth dams. Not suitable for clean well-graded sands or silty uniform sands.	6 to 8	6 coverages	3-wheel roller for compaction of fine-grained soil; weights from 5 to 6 tons for materials of low plasticity to 10 tons for materials of high plasticity.	

TABLE 5 (continued)
Compaction Equipment and Methods

Equipment Type	Applicability	Requirements for Compaction of 95 to 100 Percent Standard Proctor Maximum Density			Possible Variations in Equipment
		Compacted Lift Thickness, in.	Passes or Coverages	Dimensions and Weight of Equipment	
Vibrating Sheetsfoot Rollers	For coarse-grained soils sand-gravel mixtures	8 to 12	3 to 5	1 to 20 tons ballasted weight. Dynamic force up to 20 tons.	May have either fixed or variable cyclic frequency.
Vibrating Smooth Drum Rollers	For coarse-grained soils sand-gravel mixtures - rock fills	6 to 12 (soil) to 36 (rock)	3 to 5 4 to 6	- do -	- do -
Vibrating Baseplate Compactors	For coarse-grained soils with less than about 12 percent passing No. 200 sieve. Best suited for materials with 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	8 to 10	3 coverages	Single pads or plates should weigh no less than 200 lbs. May be used in tandem where working space is available. For clean coarse-grained soil, vibration frequency should be no less than 1,600 cycles per minute.	Vibrating pads or plates are available, hand-propelled, single or in gangs, with width of coverage from 1-1/2 to 15 ft. Various types of vibrating-drum equipment should be considered for compaction in large areas.
Crawler Tractor	Best suited for coarse-grained soils with less than 4 to 8 percent passing No. 200 sieve, placed thoroughly wet.	6 to 10	3 to 4 coverages	Vehicle with "Standard" tracks having contact pressure not less than 10 psi.	Tractor weight up to 85 tons.
Power Tamper or Rammer	For difficult access, trench backfill. Suitable for all inorganic soils.	4 to 6 in. for silt or clay, 6 in. for coarse-grained soils.	2 coverages	30-lb minimum weight. Considerable range is tolerable, depending on materials and conditions.	Weights up to 250 lbs., foot diameter 4 to 10 in.

Adjust laboratory maximum standard density (from moisture-density relations test, see DM-7.01 Chapter 3) to provide a reference density to which field density test results (with oversize) can be compared. Use the following equations to adjust the laboratory maximum dry density and optimum moisture content to values to which field test data (with oversize particles) may be compared.

$$\gamma_{max} = \frac{1 - (0.05)(F)}{\frac{F}{162} + \frac{1-F}{\gamma_l}}$$

where: γ_{max} = adjusted maximum dry density pcf

γ_l = laboratory maximum dry density without oversize, pcf

F = fraction of oversize particles by weight (from field density test)

$$w_j = F(w_g) + (1-F)w_o$$

where: w_j = adjusted optimum moisture content

w_g = moisture content of oversize (from field data)

w_o = laboratory optimum moisture content without oversize

The density of oversize material is assumed as 162 pcf, obtained from bulk specific gravity 2.60, multiplied by 62.4.

This method is considered suitable when the weight of oversize is less than 60% by weight, for well-graded materials. For poorly graded materials, further adjustment may be appropriate. This method is modified after that described in Reference 4, Suggested Method for Correcting Maximum Density and Optimum Moisture Content of Compacted Soils for Oversize Particles, by McLeod; also see Reference 5, Scalping and Replacement Effects on the Compaction Characteristics of Earth-Rock Mixtures, by Donaghe and Townsend.

Section 4. EMBANKMENT COMPACTION CONTROL

1. GROUND PREPARATION.

(1) Strip all organics and any other detrimental material from the surface. in prairie soils this may amount to removal of 2 or 3 inches of topsoil, and in forest covered land between 2 and 5 or more feet. Only the heavy root mat and the stumps need be removed, not the hair-like roots.

(2) Remove subsurface structures or debris which will interfere with the compaction or the specified area use.

(3) Scarify the soil, and bring it to optimum moisture content.

(4) Compact the scarified soil to the specified density.

2. FIELD TEST SECTION. By trial, develop a definite compaction procedure (equipment, lift thickness, moisture application, and number of passes) which will produce the specified density. Compaction cannot be controlled adequately by spot testing unless a well defined procedure is followed.

3. REQUIREMENTS FOR CONTROL TESTS. Perform in-place field density tests plus sufficient laboratory moisture-density tests to evaluate compaction. For high embankments involving seepage, settlement, or stability, perform periodic tests for engineering properties of density test samples, e.g., permeability tests, shear strength tests. See DM-7.1, Chapter 3 for laboratory moisture density test procedures and DM-7.1, Chapter 2 for field density test methods.

a. Number of Field Density Tests. Specify the following minimum test schedule:

(1) One test for every 500 cu yd of material placed for embankment construction.

(2) One test for every 500 to 1,000 cu yd of material for canal or reservoir linings or other relatively thin fill sections.

(3) One test for every 100 to 200 cu yd of backfill in trenches or around structures, depending upon total quantity of material involved.

(4) At least one test for every full shift of compaction operations on mass earthwork.

(5) One test whenever there is a definite suspicion of a change in the quality of moisture control or effectiveness of compaction.

b. Field Density Test Methods. See DM-7.1, Chapter 2, for field density test methods.

Proofrolling (spotting soft spots with a rubber-tired roller or any loaded earth-moving equipment) may be used in conjunction with density testing, but is practical only for extensive earthwork or pavement courses.

c. Laboratory Compaction Tests. Prior to important earthwork operations, obtain a family of compaction curves representing typical materials. Ideally, this family will form a group of parallel curves and each field density test will correspond to a specific compaction curve.

During construction obtain supplementary compaction curves on field density test samples, approximately one for every 10 or 20 field tests, depending on the variability of materials.

4. ANALYSIS OF CONTROL TEST DATA. Compare each field determination of moisture and density with appropriate compaction curve to evaluate conformance to requirements.

a. Statistical Study. Overall analysis of control test data will reveal general trends in compaction and necessity for altering methods. Inevitably, a certain number of field determinations will fall below specified density or outside specified moisture range. Tabulate field tests, noting the percentage difference between field density and laboratory maximum density and between field moisture and optimum.

b. Moisture Control. Close moisture control is evidenced if two-thirds of all field values fall in a range +/- 1 percent about the median moisture content specified. Erratic moisture control is evidenced if approximately two-thirds of all field values fall in a range +/- 3 percent about the median moisture content specified. To improve moisture control, blend materials from wet and dry sections of borrow area.

c. Compactive Effort. Suitable compaction methods are being utilized if approximately two-thirds of all field densities fall in a range of +/- 3 percent about the percent maximum density required. Insufficient or erratic compaction is evidenced if approximately two-thirds of all field values fall in a range of +/- 5 percent about the percent maximum density required. To improve compaction, consider methods for more uniform moisture control, alter the number of coverages, weights, or pressures of compaction equipment.

d. Overcompaction. A given compactive effort yields a maximum dry density and a corresponding optimum moisture content. If the compactive effort is increased, the maximum dry density increases but the corresponding optimum moisture content decreases. Thus, if the compactive effort used in the field is higher than that used in the laboratory for establishing the moisture density relationship, the soil in the field may be compacted above its optimum moisture content, and the strength of the soil may be lower even though it has been compacted to higher density. This is of particular concern for high embankments and earth dams. For further guidance see Reference 6, Stabilization of Materials by Compaction, by Turnbull and Foster.

5. INDIRECT EVALUATION OF COMPACTION IN DEEP FILLS. The extent of compaction accomplished is determined by comparing the results from standard penetration tests and cone penetration tests before and after treatment (DM-7.1, Chapter 2).

6. PROBLEM SOILS. The compaction of high volume change soils requires special treatment. See DM-7.3, Chapter 3.

Section 5. BORROW EXCAVATION

1. BORROW PIT EXPLORATION

a. Extent. The number and spacing of borings or test pits for borrow exploration must be sufficient to determine the approximate quantity and quality of construction materials within an economical haul distance from the project. For mass earthwork, initial exploration should be on a 200-foot grid. If variable conditions are found during the initial explorations, intermediate borings or test pits should be done. Explorations should develop the following information:

(1) A reasonably accurate subsurface profile to the anticipated depth of excavation.

(2) Engineering properties of each material considered for use.

(3) Approximate volume of each material considered for use.

(4) Water level.

(5) Presence of salts, gypsums, or undesirable minerals.

(6) Extent of organic or contaminated soils, if encountered.

2. EXCAVATION METHODS.

a. Equipment. Design and efficiency of excavation equipment improves each year. Check various construction industry publications for specifications.

b. Ripping and Blasting. Determine rippability of soil or rock by borings (RQD and core recovery, see DM-7.01, Chapters 1 and 2), geophysical exploration, and/or trial excavation.

3. UTILIZATION OF EXCAVATED MATERIALS. In the process of earthmoving there may be a reduction of the volume ("shrinkage") because of waste and densification, or an increase of volume ("swell") in the case of rock or dense soils, because the final density is less than its original density.

a. Borrow Volume. Determine total borrow volume, V+B, required for compacted fill as follows:

$$V_B = \left(\frac{\gamma_F}{\gamma_B} \cdot V_F\right) + \frac{W_L}{\gamma_B}$$

where: γ_F = dry unit weight of fill

γ_B = dry unit weight of borrow

V_F = required fill volume

W_L = weight lost in stripping, waste, oversize and transportation

(1) Compacted Volume. The volume of borrow soil required should be increased according to the volume change indicated above. A "shrinkage" factor of 10 to 15 percent may be used for estimating purposes.

(2) Exclusions. A large percentage of cobble size material will increase the waste, because sizes larger than 3 inches are generally excluded from compacted fill.

b. Rock Fill.

(1) Maximum Expansion. Maximum expansion ("swell") from in situ conditions to fill occurs in dense, hard rock with fine fracture systems that

breaks into uniform sizes. Unit volume in a quarry will produce approximately 1.5 volumes in fill.

(2) Minimum Expansion. Minimum expansion occurs in porous, friable rock that breaks into broadly graded sizes with numerous spalls and fines. Unit volume in quarry will produce approximately 1.1 volumes in fill.

Section 6. HYDRAULIC AND UNDERWATER FILLS

1. GENERAL. Where large quantities of soil must be transported and ample water is available, hydraulic methods are economical. The choice of methods for placing hydraulic fill is governed by the type of equipment available, accessibility of borrow, and environmental regulations; see Table 6 (Reference 7, Control for Underwater Construction, by Johnson, et al.). Removal or placement of soil by hydraulic methods must conform to applicable water pollution control regulations.

2. PLACEMENT METHODS. Placement, either under water or on land, should be done in a manner that produces a usable area with minimum environmental impact.

a. Deep Water Placement (over 75 feet). Most deep water placement is by bottom dump scows and is unconfined, with no control on turbidity, except by the rate of dumping.

b. Shallow Water Placement. Placement by pipeline, by mechanical equipment, or by side dumping from deck scows are the most common methods in shallow water. Sheet pile containment, silt "curtains", or dikes are required to minimize lateral spreading and environmental impact. Where lateral spreading is not desired and steeper side slopes are needed, control the method of placement or use a mixed sand and gravel fill material. With borrow containing about equal amounts of sand and gravel, underwater slopes as steep as 1:3 or 1:2-3/4 may be achieved by careful placement. To confine the fill, provide berms or dikes of the coarsest available material or stone on the fill perimeter. Where rock is placed underwater, sluice voids with sand to reduce compressibility and possible loss of material into the rock.

c. Land Placement. On land, hydraulic fills are commonly placed by pipeline or by mechanical procedures (i.e. clam shell, dragline, etc.). Dikes with adjustable weirs or drop inlets to control the quality of return water are used for containment.

3. PERFORMANCE OF HYDRAULIC FILLS.

a. Coarse-Grained Fills. The most satisfactory hydraulically placed fills are those having less than 15 percent non-plastic fines or 10 percent plastic fines because they cause the least turbidity during placement, drain faster, and are more suitable for structural support than fine-grained material. Relative densities of 50 to 60 percent can be obtained without compaction. Bearing values are in the range of 500 to 2000 pounds per square foot depending on the level of permissible settlement. Density, bearing and

TABLE 6
Methods of Fill Placement Underwater

Methods	Characteristics
Bottom-dump scows	1. Limited to minimum depths of about 15 ft. because of scow and tug drafts. 2. Rapid; quick discharge entraps air and minimizes segregation.
Deck scows	1. Usable in shallow water. 2. Unloading is slow 9 by dozer, clamshell, or hydraulic jets. 3. Inspection of material being placed may be difficult.
Dumping at land edge of fill and pushing material material into water by bulldozer	1. Fines in material Placed below water tend to separate and accumulate in front of advancing fill. 2. Work arrangement should result in central portion being in advance of side portions to displace sideways any soft bottom materials. 3. In shallow water, bulldozer blade can shove materials downward to assist displacement of soft materials.

resistance to seismic liquefaction may be increased substantially by vibroprobe methods. See DM-7.3, Chapter 2.

b. Fine-Grained Fills. Hydraulically placed, bottom silts and clays such as produced by maintenance dredging will initially be at very high water contents. Depending on measures taken to induce surface drainage, it will take approximately 2 years before a crust sufficient to support light equipment is formed and the water content of the underlying materials approaches the liquid limit. Placing 1 to 3 feet of additional granular borrow will improve these areas rapidly so that they can support surcharge fills, with or without vertical sand drains to accelerate consolidation. Care must be exercised in applying the surcharge so that the shear strength of the soil is not exceeded.

4. CONSOLIDATION OF HYDRAULIC FILLS. If the coefficient of permeability of a hydraulic fill is less than 0.002 feet per minute, the consolidation time for the fill will be long and prediction of the behavior of the completed fill will be difficult. For coarse-grained materials, fill consolidation and strength build-up will be rapid and reasonable strength estimates can be made. Where fill and/or foundation soils are fine-grained, it may be desirable to monitor settlement and pore water pressure dissipation if structures are planned. Settlement plates may be placed both on the underlying soil and within the fill to observe settlement rates and amounts.

REFERENCES

1. Sherard, J.L., Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams, Technical Memorandum 645, U.S. Department of the Interior, Bureau of Reclamation.

2. Sherard, J.L., Dunnigan, L.P., Becker, R.S., and Steele, E.S., Pinhole Test for Identifying Dispersive Soils, Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT1, 1976.

3. Gordori, B.B., Miller, R.K., Control of Earth and Rockfill for Oroville Dam, Journal of the Geotechnical Engineering Division, ASCE, Vol. 92, No. SM3, 1966.

4. McLeod, N.W., Suggested Method for Correcting Maximum Density and Optimum Moisture Content of Compacted Soils for Oversize Particles, Special Procedures for Testing Soil and Rock for Engineering Purposes, ASTM STP 479, ASTM, 1970.

5. Donaghe, R.T., and Townsend, F.C., Scalping and Replacement Effects on the Compaction Characteristics of Earth-Rock Mixtures, Soil Specimen Preparation for Laboratory Testing, ASTM STP 599, ASTM, 1976.

6. Turnbull, W.J. and Foster, C.R., Stabilization of Materials by Compaction, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 82, No. SM2, 1956.

7. Johnson, S.J., Compton, J.R., and Ling, S.C., Control for Underwater Construction, Underwater Sampling, Testing, and Construction Control, ASTM STP 501, ASTM, 1972.

8. Naval Facilities Engineering Command

   Design Manuals (DM)
   - DM- 5.04 Pavements
   - DM-21.03 Flexible Pavement Design for Airfields
   - DM-26.03 Coastal Sedimentation and Dredging
   - DM-38.02 Dredging Equipment

   Copies may be obtained from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120.

PAGE 58 INTENTIONALLY BLANK

CHAPTER 3. ANALYSIS OF WALLS AND RETAINING STRUCTURES

Section 1. INTRODUCTION

1. SCOPE. Methods of determining earth pressures acting on walls and retaining structures are summarized in this chapter. Types of walls considered include concrete retaining walls and gravity walls that move rigidly as a unit, braced or tied bulkheads of thin sheeting that deflect according to the bracing arrangement, and double-wall cofferdams of thin sheeting to confine earth or rock fill.

2. RELATED CRITERIA. Additional criteria relating to the design and utilization of walls appear in the following sources:

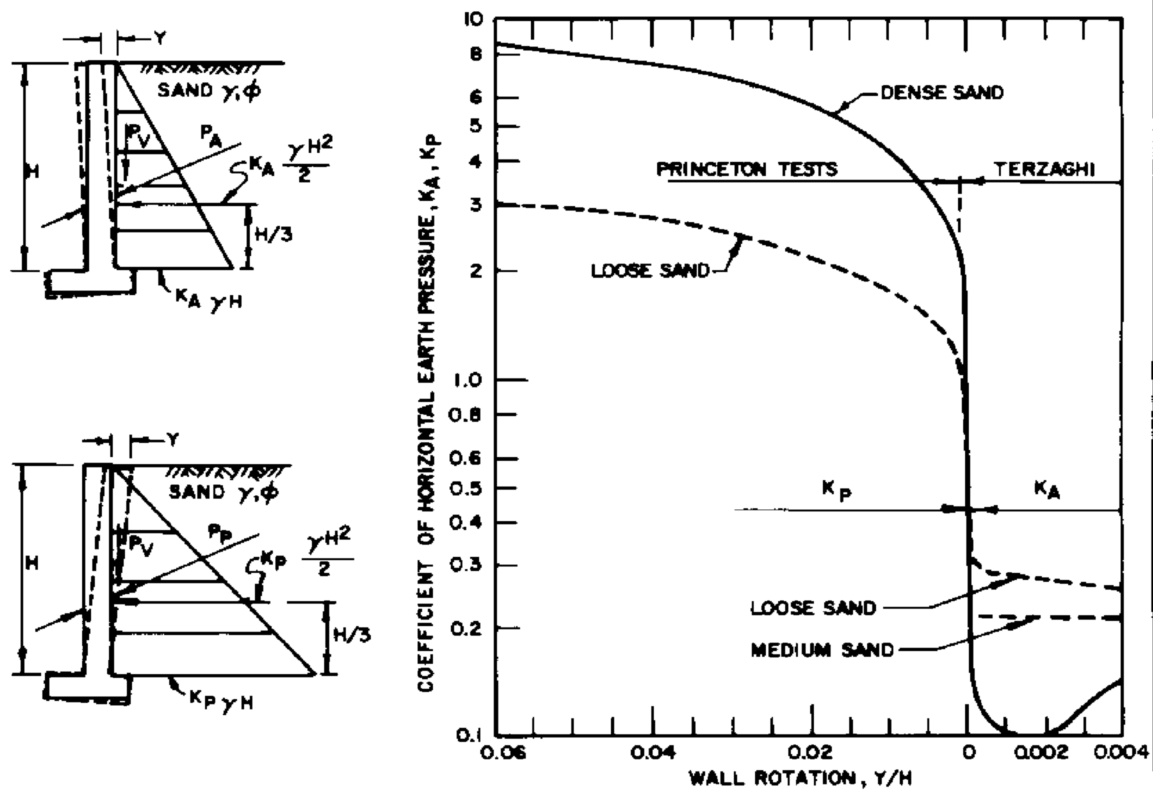
Subject	Source
Application of Bulkheads and Cofferdams to Waterfront Construction	NAVFAC DM-25.04
Structural Design of Retaining Walls	NAVFAC DM-2 Series

Section 2. COMPUTATION OF WALL PRESSURES

1. CONDITIONS. The pressure on retaining walls, bulkheads, or buried anchorages is a function of the relative movement between the structure and the surrounding soil.

a. <u>Active State</u>. Active earth pressure occurs when the wall moves away from the soil and the soil mass stretches horizontally sufficient to mobilize its shear strength fully, and a condition of plastic equilibrium is reached. (See Figure 1 from Reference 1, <u>Excavations and Retaining Structures</u>, by the Canadian Geotechnical Society.) The ratio of the horizontal component or active pressure to the vertical stress caused by weight of soil is the active pressure coefficient (K+a,). The active pressure coefficient as defined above applies only to cohesionless soils.

b. <u>Passive State</u>. Passive earth pressure occurs when a soil mass is compressed horizontally, mobilizing its shear resistance fully (see Figure 1). The ratio of the horizontal component of passive pressure to the vertical stress caused by the weight of the soil is the passive pressure coefficient (K+p,). The passive coefficient, as defined here, applies only the cohesionless soil. A soil mass that is neither stretched nor compressed is said to be in an at-rest state. The ratio of lateral stress to vertical stress is call the at-rest coefficient (K+o,).



MAGNITUDES OF WALL ROTATION TO REACH FAILURE

SOIL TYPE AND CONDITION	ROTATION Y/H*	
	ACTIVE	PASSIVE
DENSE COHESIONLESS	.0005	.002
LOOSE COHESIONLESS	.002	.006
STIFF COHESIVE	.01	.02
SOFT COHESIVE	.02	.04

* Y = HORIZONTAL DISPLACEMENT
H = HEIGHT OF THE WALL

FIGURE 1
Effect of Wall Movement on Wall Pressures

2. COMPUTATION OF ACTIVE AND PASSIVE PRESSURES. See Figure 2 for formulas for active and passive pressures for the simple case on a frictionless vertical face with horizontal ground surface. Three basic conditions required for validity of the formulas are listed in Figure 2. Under these conditions the failure surface is a plane and the formulas represent pressures required for equilibrium of the wedge shaped failure mass.

The intensity of pressures applied depends on wall movements, as these control the degree of shear strength mobilization in surrounding soil. (See Figure 1 for the magnitude of the movement necessary for active condition to exist.) Wall friction and wall vertical movements also affect the passive and active pressures.

The effect of wall friction on active pressures is small and ordinarily is disregarded except in case of a settling wall where it can be very significant. The effect of wall friction on passive pressures is large, but definite movement is necessary for mobilization of wall friction. (See Table 1 for typical ultimate friction factors and adhesion between wall and backfill.) In the absence of specific test data, use these values in computations that include effects of wall friction.

Unless a wall is settling, friction on its back acts upward on the active wedge (angle [delta] is positive, see Figure 5), reducing active pressures. Generally, wall friction acts downward against the passive wedge (angle [delta] is negative), resisting its upward movement and increasing passive pressures.

a. Uniform Backfill, No Groundwater. Compute active and passive pressures by methods from Figure 2.

b. Sloping Backfill, No Groundwater, Granular Soil, Smooth Wall. Compute active and passive pressures by methods from Figure 3. Use Figure 4 to determine the position of failure surface for active and passive wedge.

c. Sloping Wall, Granular Soil With Wall Friction. Use Figure 5 (Reference 2, Tables for the Calculation of the Passive Pressure, Active Pressure and Bearing Capacity of Foundations, by Caquot and Kerisel) to compute active and passive earth pressure coefficients.

d. Sloping Backfill, Granular Soil with Wall Friction. Use Figure 6 (Reference 2) to compute active and passive earth pressure coefficient.

e. Uniform Backfill, Static Groundwater. Compute active earth and water pressures by formulas in Figure 7.

f. General Formula for Coefficients of Passive and Active Earth Pressure. Use Figure 8 for sloping wall with friction and sloping backfill.

g. Stratified Backfill, Sloping Groundwater Level. When conditions include layered soil, irregular surcharge, wall friction, and sloping groundwater level, determine active pressures by trial failure wedge. (See Figure 7.) Trial wedge is bounded by a straight failure plane or a series of straight segments at different inclination in each stratum. Commence the analysis with failure plane oriented at the angle shown in Figure 4.

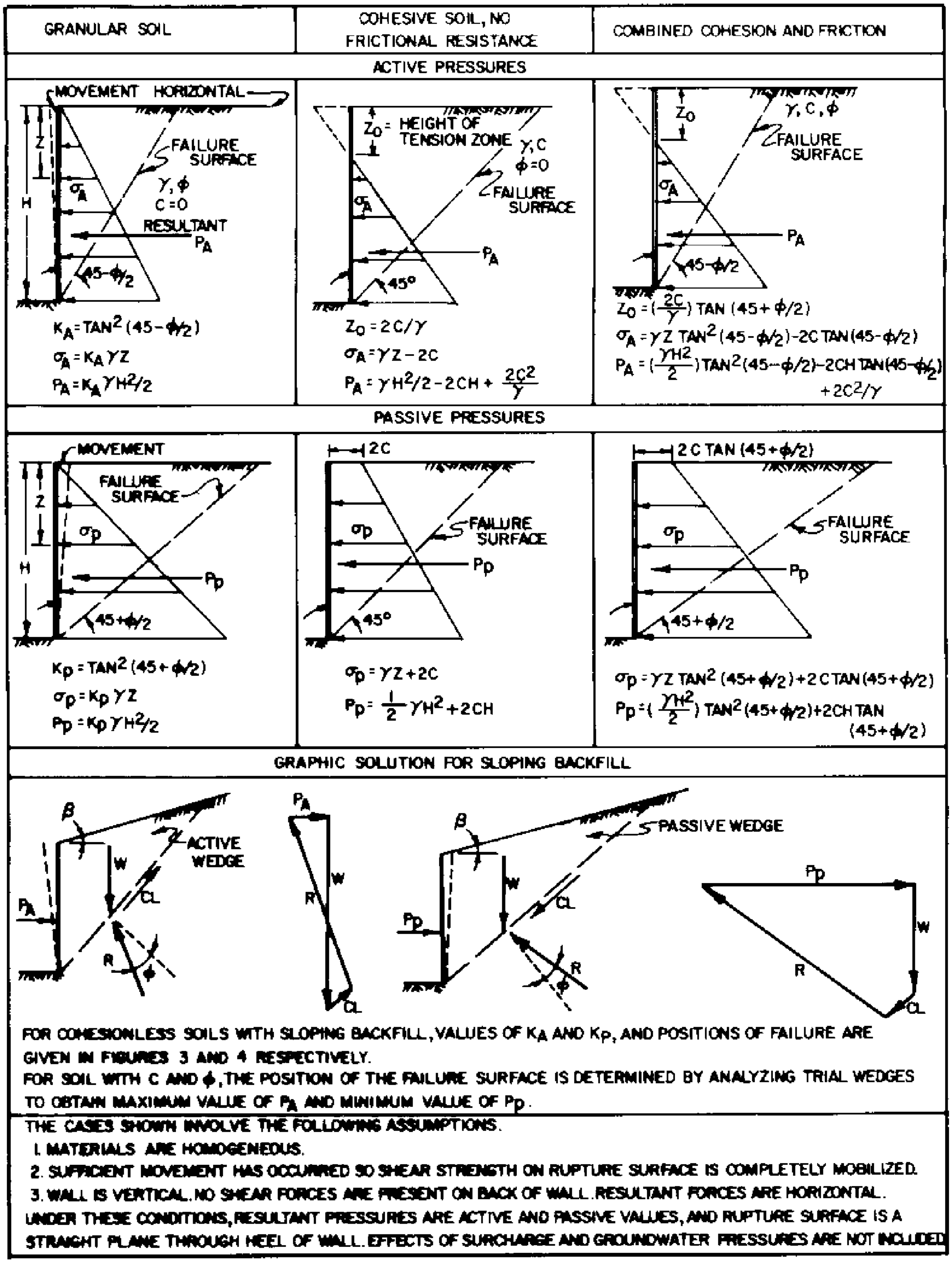
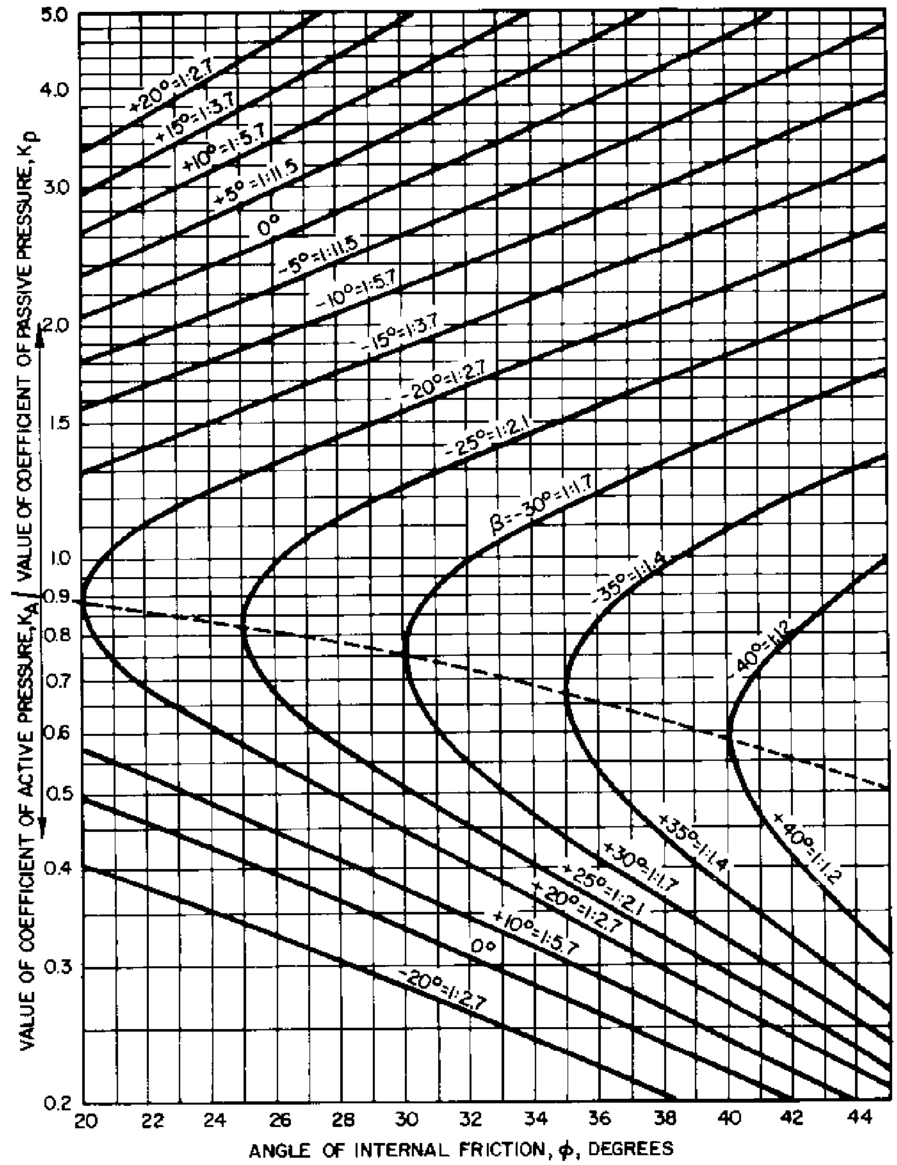


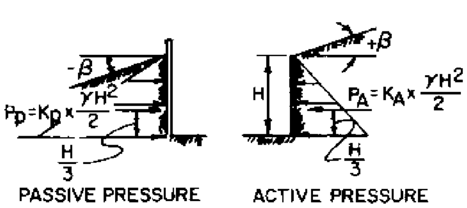
FIGURE 2
Computation of Simple Active and Passive Pressures

TABLE 1
Ultimate Friction Factors and Adhesion for Dissimilar Materials

Interface Materials	Friction factor, tan [delta]	Friction angle [delta] degrees
Mass concrete on the following foundation materials:		
Clean sound rock	0.70	35
Clean gravel, gravel-sand mixtures, coarse sand	0.55 to 0.60	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	0.45 to 0.55	24 to 29
Clean fine sand, silty or clayey fine to medium sand	0.35 to 0.45	19 to 24
Fine sandy silt, nonplastic silt	0.30 to 0.35	17 to 19
Very stiff and hard residual or preconsolidated clay	0.40 to 0.50	22 to 26
Medium stiff and stiff clay and silty clay	0.30 to 0.35	17 to 19
(Masonry on foundation materials has same friction factors.)		
Steel sheet piles against the following soils:		
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30	17
Silty sand, gravel or sand mixed with silt or clay	0.25	14
Fine sandy silt, nonplastic silt	0.20	11
Formed concrete or concrete sheet piling against the following soils:		
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	0.40 to 0.50	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	0.30 to 0.40	17 to 22
Silty sand, gravel or sand mixed with silt or clay	0.30	17
Fine sandy silt, nonplastic silt	0.25	14
Various structural materials:		
Masonry on masonry, igneous and metamorphic rocks:		
Dressed soft rock on dressed soft rock	0.70	35
Dressed hard rock on dressed soft rock	0.65	33
Dressed hard rock on dressed hard rock	0.55	29
Masonry on wood (cross grain)	0.50	26
Steel on steel at sheet pile interlocks	0.30	17

Interface Materials (Cohesion)	Adhesion c+a, (psf)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300





$$K_A = \left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi - \cos\phi\tan\beta)}}\right]^2; \quad K_P = \left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi + \cos\phi\tan\beta)}}\right]^2$$

K_A & K_P = COEFFICIENTS FOR COULOMB'S EQUATION FOR ACTIVE AND PASSIVE EARTH PRESSURE (NO SHEAR STRESS ON VERTICAL PLANES).

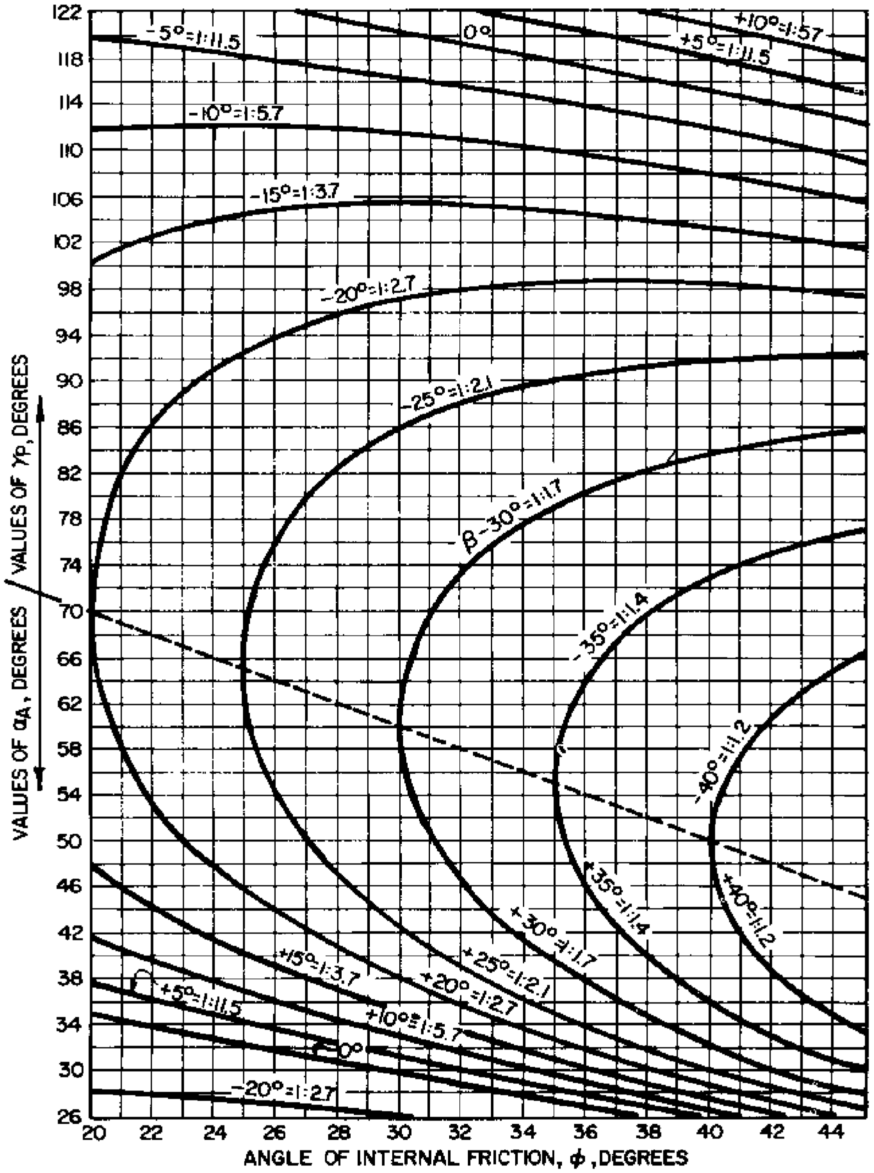
P_A = ACTIVE RESULTANT $\quad$ ϕ = ANGLE OF INTERNAL FRICTION

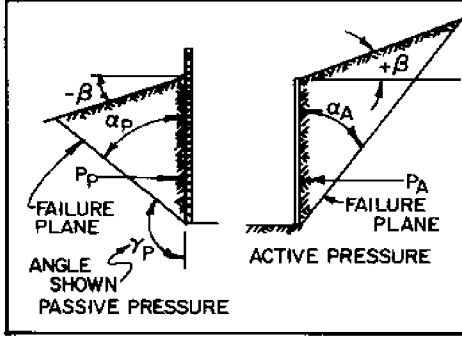
P_P = PASSIVE RESULTANT $\quad$ β = SLOPE ANGLE

γ = UNIT WEIGHT OF SOIL

H = HEIGHT OF WALL

FIGURE 3
Active and Passive Coefficients, Sloping Backfill
(Granular Soils)





$$\text{COT } \alpha_A = \text{TAN } \phi + \sqrt{1 + \text{TAN}^2 \phi - \frac{\text{TAN } \beta}{\text{SIN } \phi \text{ COS } \phi}}$$

$$\text{COT } \alpha_P = -\text{TAN } \phi + \sqrt{1 + \text{TAN}^2 \phi - \frac{\text{TAN } \beta}{\text{SIN } \phi \text{ COS } \phi}}$$

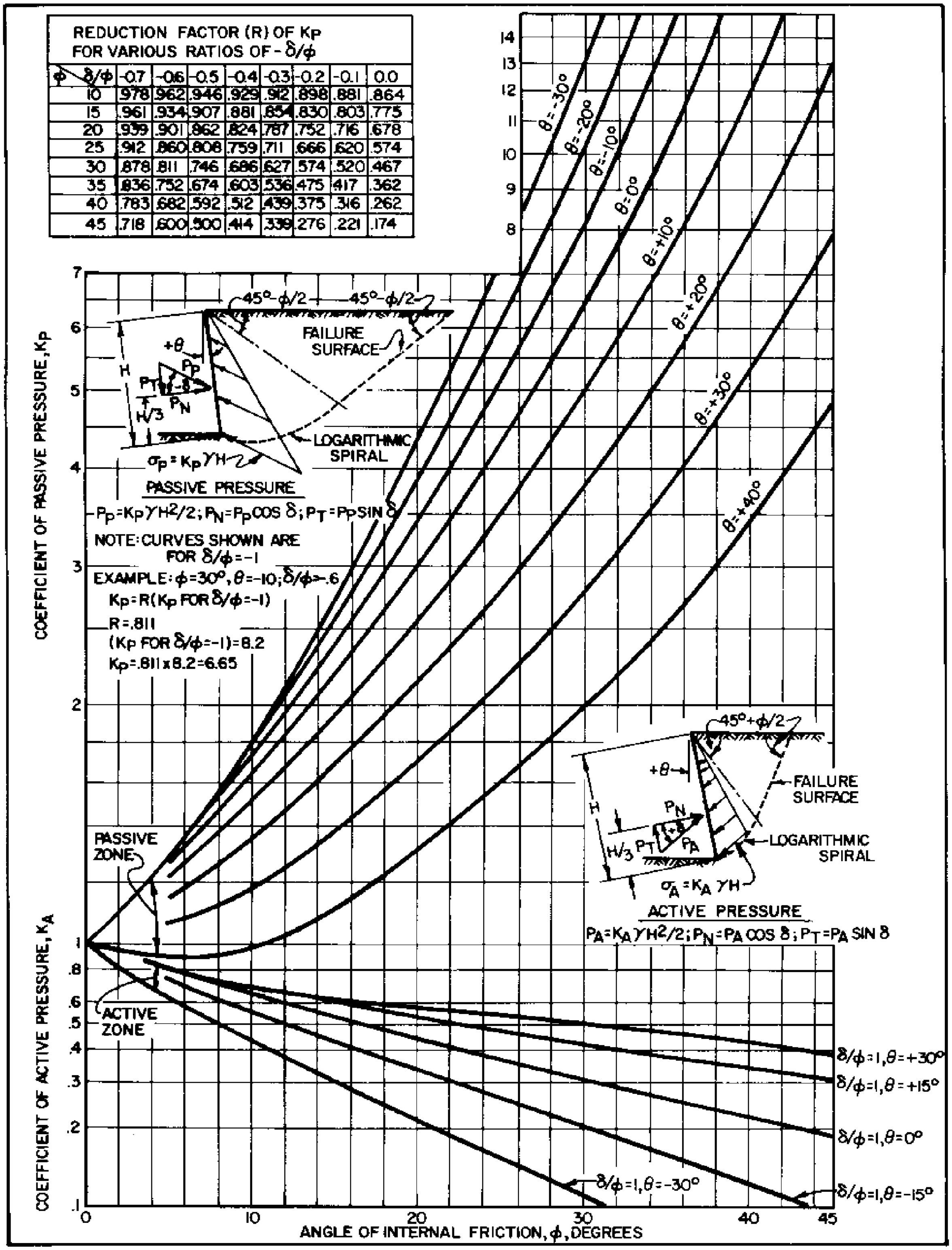
α_A & α_P = ANGLE BETWEEN CRITICAL FAILURE PLANE AND VERTICAL

ϕ = ANGLE OF INTERNAL FRICTION

β = SLOPE ANGLE

THE ANGLES SHOWN CORRESPOND TO THE COEFFICIENTS OF ACTIVE AND PASSIVE PRESSURE GIVEN IN FIGURE 3.

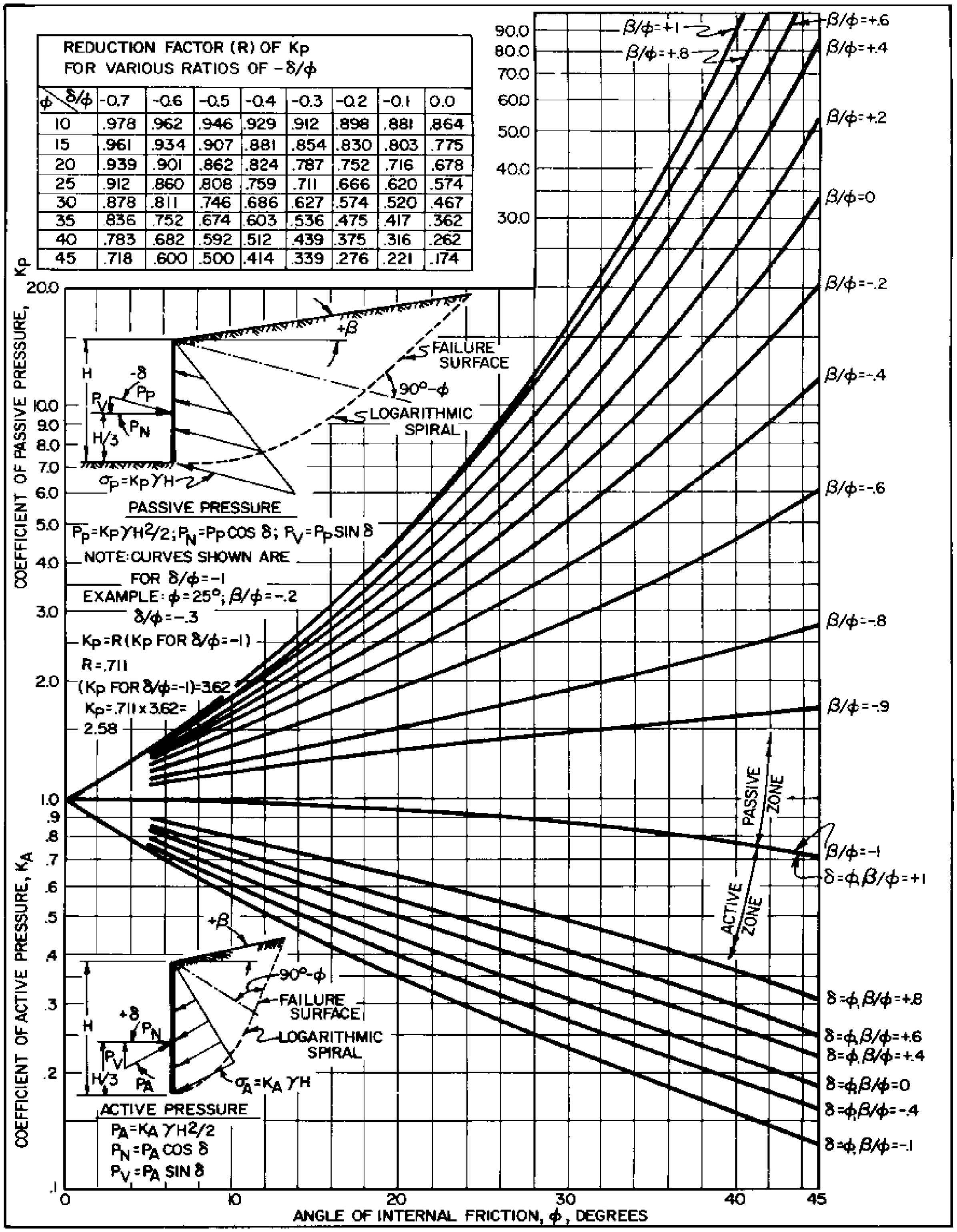
FIGURE 4
Position of Failure Surface for Active and Passive Wedges
(Granular Soils)



REDUCTION FACTOR (R) OF K_P FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

FIGURE 5
Active and Passive Coefficients with Wall Friction
(Sloping Wall)



φ \ δ/φ	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

FIGURE 6
Active and Passive Coefficients with Wall Friction
(Sloping Backfill)

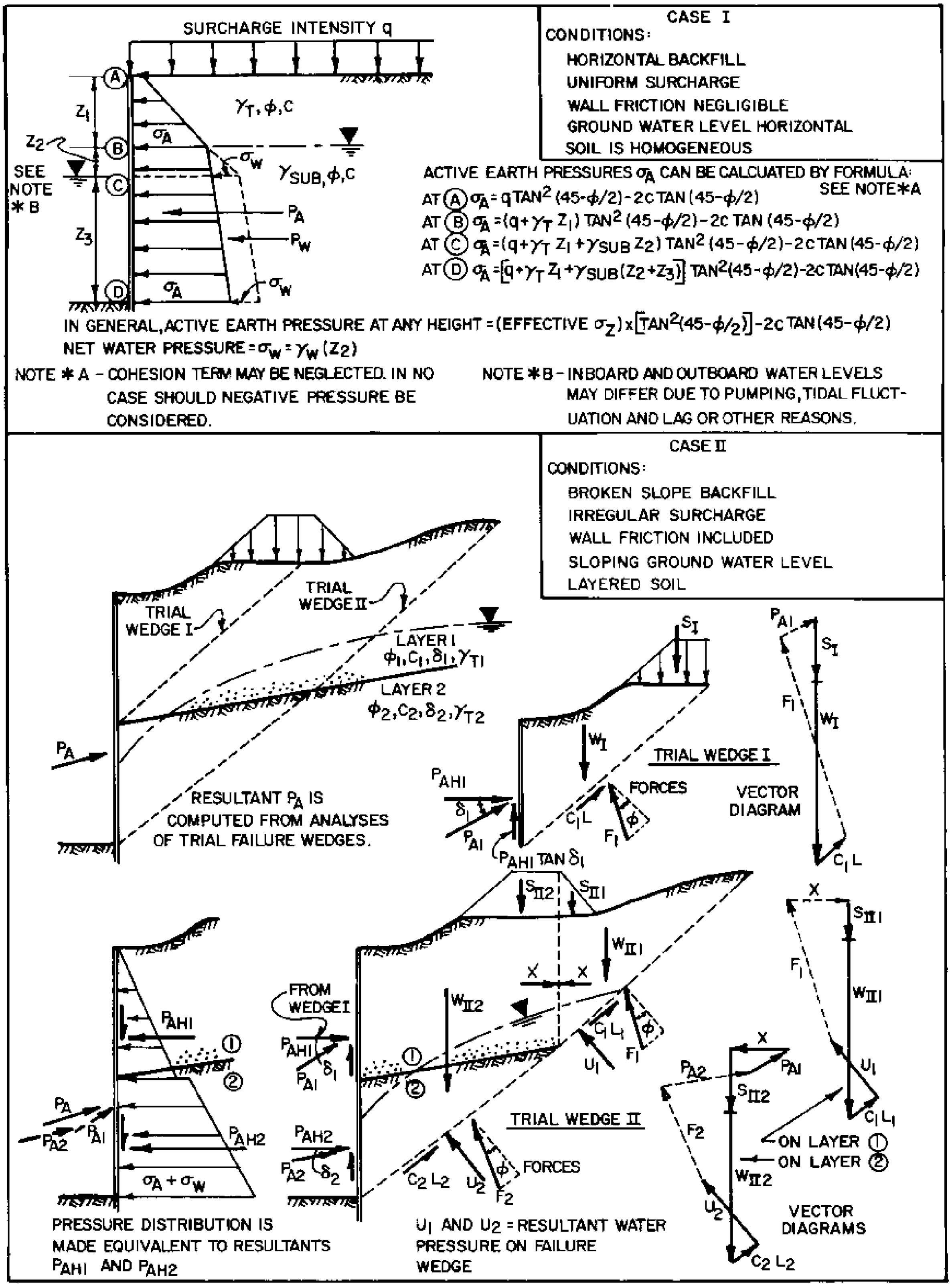
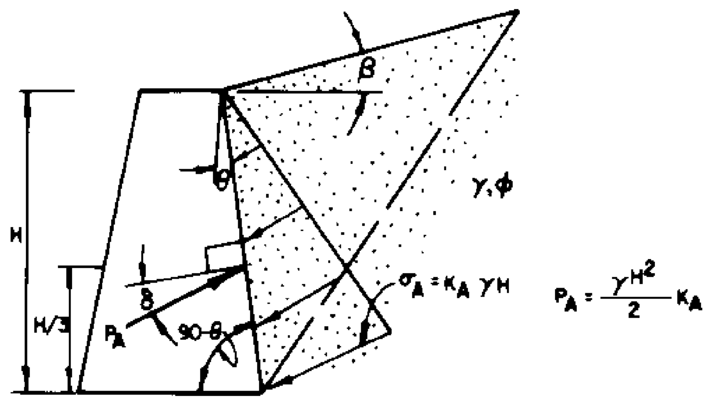
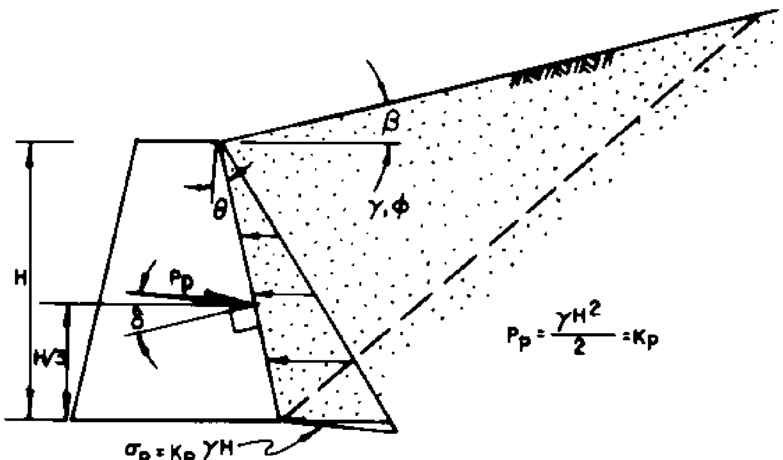


FIGURE 7
Computation of General Active Pressures



$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2\theta\cos(\theta+\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\theta+\delta)\cos(\theta-\beta)}}\right]^2}$$



$$K_p = \frac{\cos^2(\theta+\phi)}{\cos^2\theta\cos(\theta-\delta)\left[1-\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi+\beta)}{\cos(\theta-\delta)\cos(\theta-\beta)}}\right]^2}$$

K_p VALUES ARE SATISFACTORY FOR $\delta \leq \phi/3$ BUT ARE UNCONSERVATIVE FOR $\delta > \phi/3$ AND THEREFORE SHOULD NOT BE USED.

FIGURE 8
Coefficients K_A and K_p for Walls with Sloping Wall and Friction, and Sloping Backfill

Compute resultant passive force by trial failure wedge analysis. (See Figure 9). When wall friction is included, compute pressures from a failing mass bounded by a circular arc and straight plane. Determine location of passive resultant by summing moments about toe of wall of all forces on that portion of the failing mass above the circular arc. Depending on complexity of cross section, distribute passive pressures to conform to location of resultant, or analyze trial failure surfaces at intermediate heights in the passive zone. When wall friction is neglected, the trial failure surface is a straight plane. See Figure 2.

(1) Simple Cross Section. For a simple cross section behind a wall, analyze the trial failure plane extending upward from the lowest point of the active zone on the wall. Determine the location of the active resultant by summing moments of all forces on the wedge about toe of wedge. Distribute active pressures to conform to the location of resultant.

(2) Complicated Cross Section. For complicated cross sections, analyze trial wedges at intermediate heights above the base of the active zone to determine pressure distribution in more detail. Force acting on an increment of wall height equals difference in resultant forces for wedges taken from the top and bottom of that increment.

3. EFFECT OF GROUNDWATER CONDITIONS. Include in pressure computations the effect of the greatest unbalanced water head anticipated to act across the wall.

a. General Conditions. For a major structure, analyze seepage and drainage effect by flow net procedures. Uplift pressures influencing wall forces are those acting on failure surface of active or passive wedge. Resultant uplift force on failure surface determined from flow net is applied in force diagram of the failure wedge. See vector U, the resultant water force, in Figures 7 and 9.

b. Static Differential Head. Compute water pressures on walls as shown in top panel of Figure 10.

c. Rainfall on Drained Walls. For cohesionless materials, sustained rainfall Increases lateral force on wall 20 to 40 percent over dry backfill, depending on backfill friction angle. The center panel of Figure 10 (Reference 3, Contribution to the Analysis of Seepage Effects in Backfills, by Gray) shows flow net set up by rainfall behind a wall with vertical drain. This panel gives the magnitude of resultant uplift force on failure wedge for various inclinations of failure plane to be used in analysis of the active wedge.

d. Seepage Beneath Wall. See bottom panel of Figure 10 (Reference 4, The Effect of Seepage on the Stability of Sea Walls, by Richart and Schmertmann) for correction to be applied to active and passive pressures in cohesionless material for steady seepage beneath a wall.

4. SURCHARGE LOADING. For the effects of surcharge loading, see Figures 7 and 9.

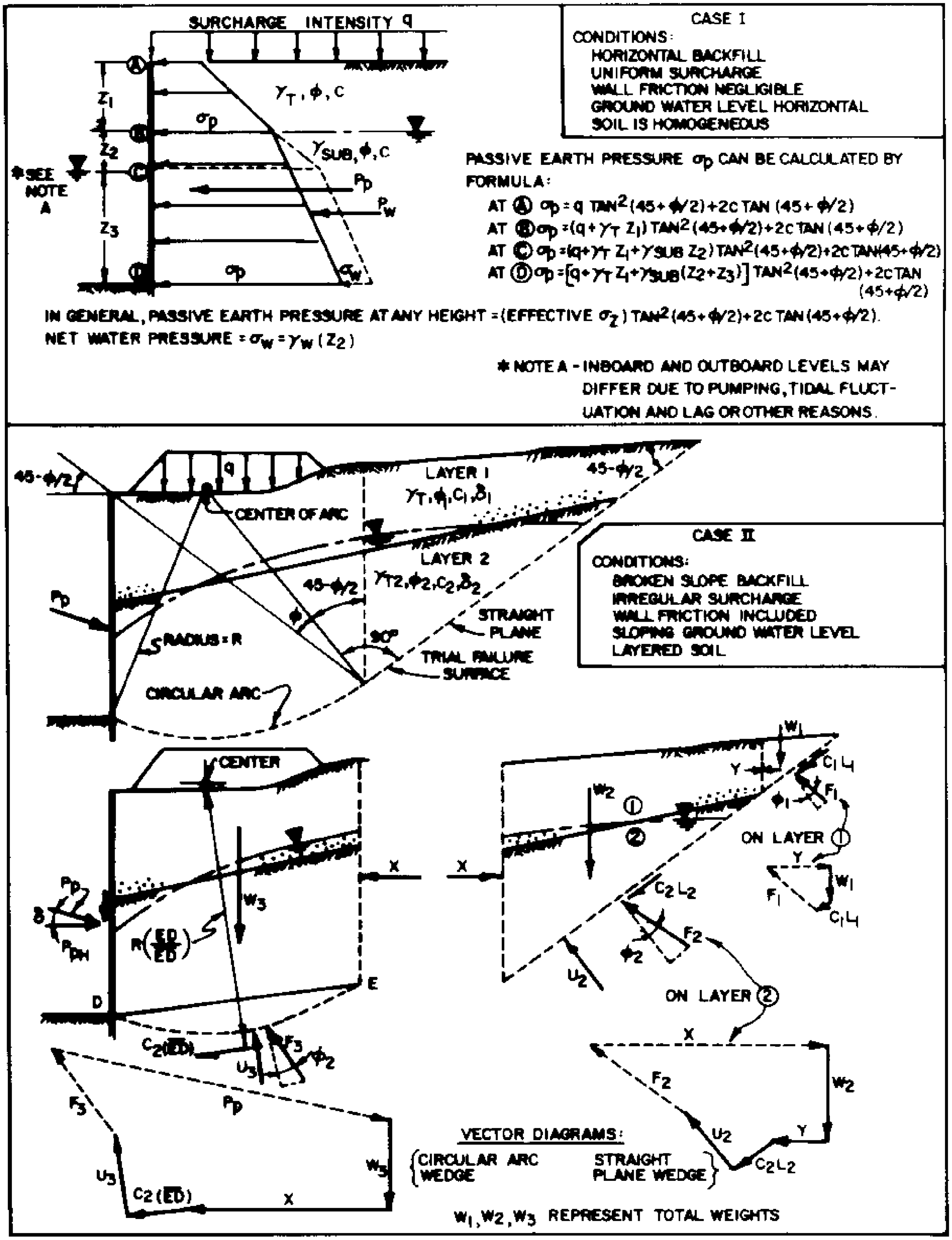


FIGURE 9
Computation of General Passive Pressures

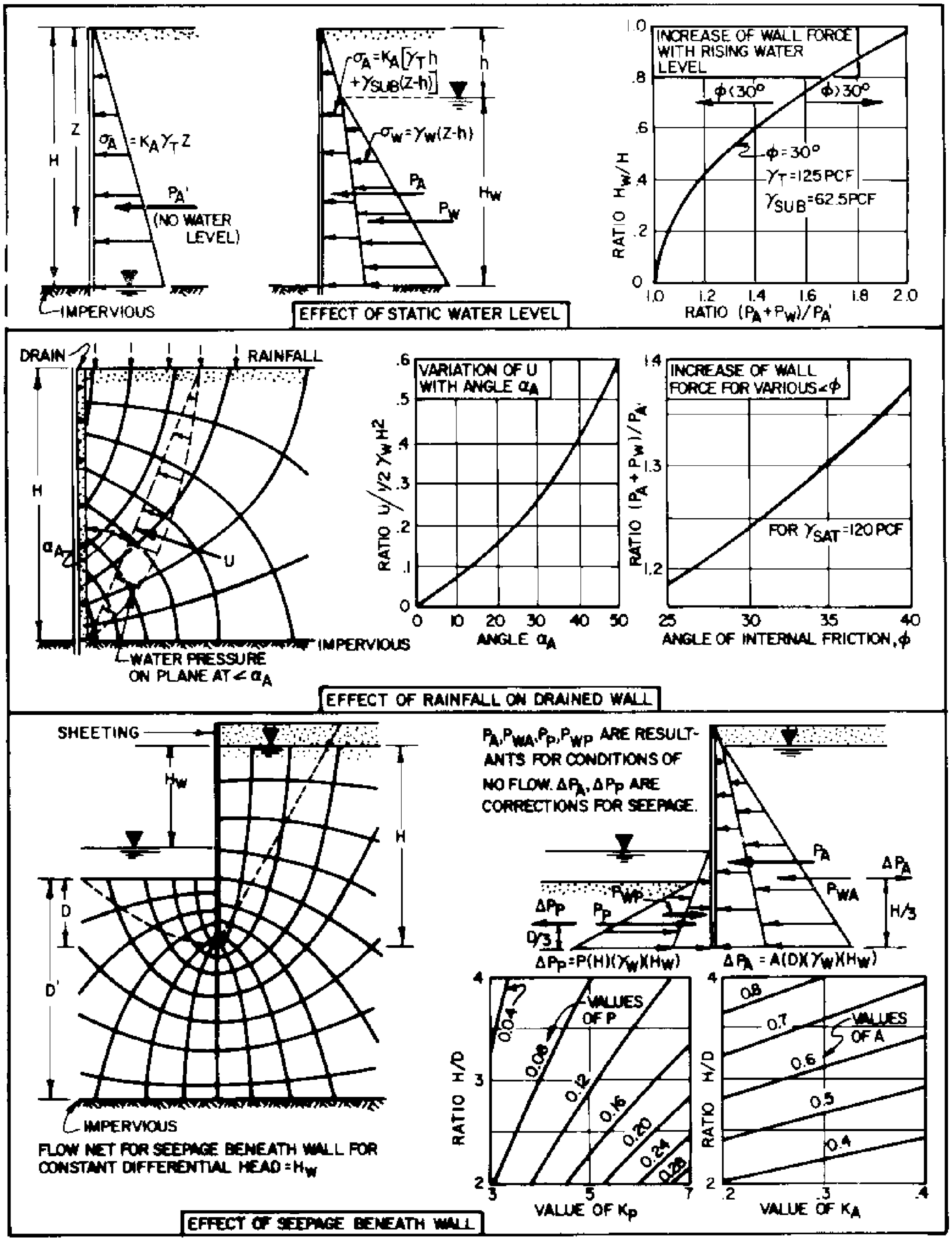


FIGURE 10
Effect of Groundwater Conditions on Wall Pressures

a. Point Load and Live Load. Use Figure 11 (Reference 5, Anchored Bulkheads, based on the work by Terzaghi) to compute lateral pressure on wall due to point load and line loads; this assumes an unyielding rigid wall and the lateral pressures are approximately double the values obtained by elastic equations. The assumption of an unyielding rigid wall is conservative and its applicability should be evaluated for each specific wall.

b. Uniform Loading Area. For uniform surcharge loading lateral stress can be computed by treating the surcharge as if it were backfill and multiplying the vertical stress at any depth by the appropriate earth pressure coefficient.

c. Uniform Rectangular Surcharge Loading. For the effect of this loading see Figure 12 (see Reference 6, Lateral Support Systems and Underpinning, 2 Volume 1, Design and Construction (Summary), by Goldberg, et al.). If the construction procedures are such that the wall will move during the application of live loads, then the pressure calculated from Figure 12 will be conservative.

d. Practical Considerations. For design purposes, it is common to consider a distributed surface load surcharge on the order of 300 psf to account for storage of construction materials and equipment. This surcharge is usually applied within a rather limited work area of about 20 feet to 30 feet from the wall and is also intended to account for concentrated loads from heavy equipment (concrete trucks, cranes, etc.) located more than about 20 feet away. If such equipment is anticipated within a few feet of the wall, it must be accounted for separately.

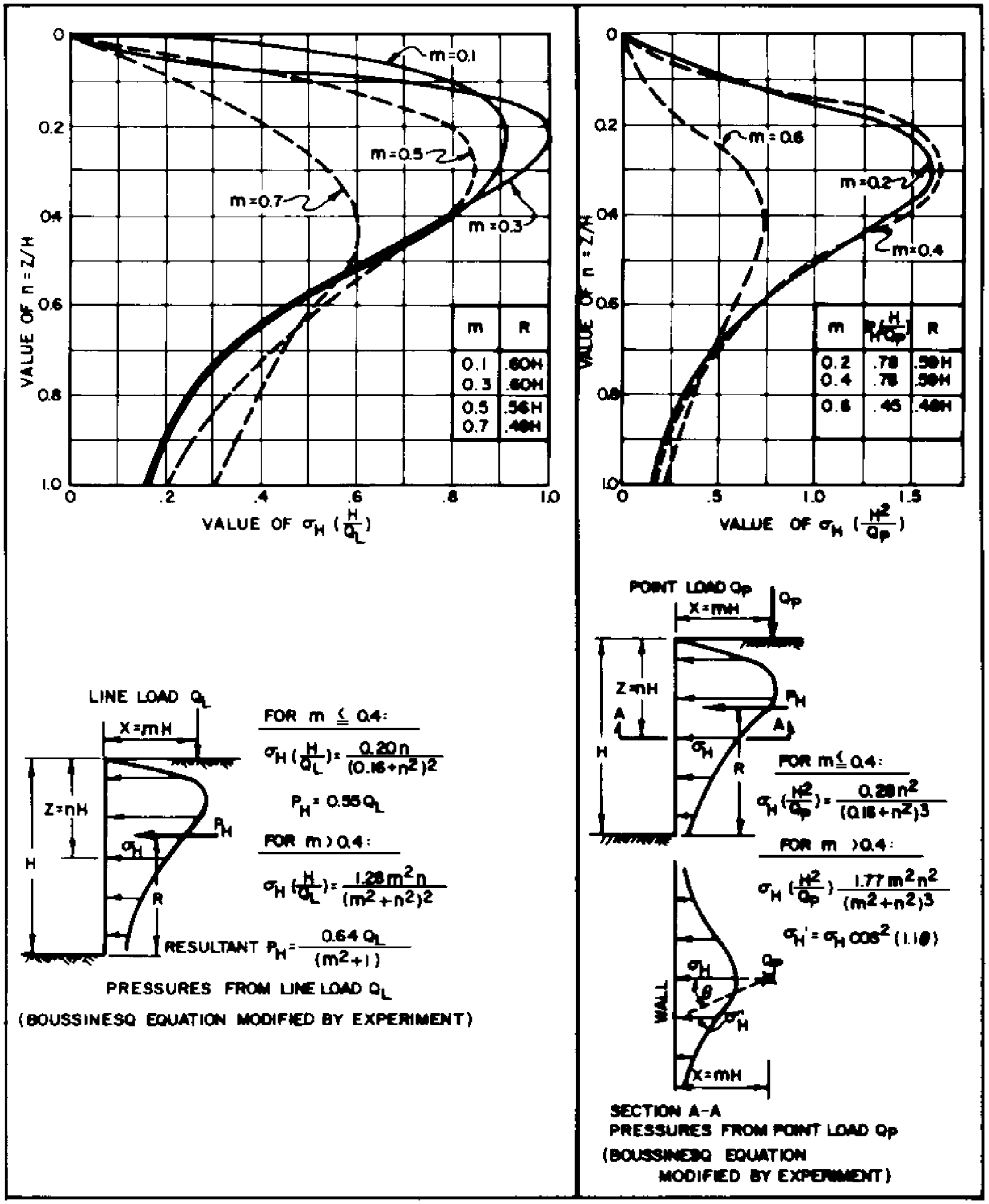
5. WALL MOVEMENT. For the effect of wall movement on the earth pressure coefficients, see Figure 1.

a. Wall Rotation. When the actual estimated wall rotation is less than the value required to fully mobilize active or passive conditions, adjust the earth pressure coefficients by using the diagram on the upper right hand corner of Figure 1. Relatively large movements are required to mobilize the passive resistance. A safety factor must be applied to the ultimate passive resistance in order to limit movements.

b. Wall Translation. Wall uniform translation required to immobilize ultimate passive resistance or active pressure is approximately equivalent to movement of top of wall based on rotation criteria given in Figure 1.

c. Internally Braced Flexible Wall. Sheeting on cuts rigidly braced at the top undergoes insufficient movement to produce fully active conditions, Horizontal pressures are assumed to be distributed in a trapezoidal diagram. (See Section 4.) The resultant force is higher than theoretical active force. For clays, the intensity and distribution of horizontal pressures depend on the stability number N+o, = [gamma] H/c. (See Section 4.)

d. Tied Back Walls. Soil movement associated with prestressed tied back walls is usually less than with internally braced flexible walls, and design pressures are higher. (See Section 4.)



m	R
0.1	.60H
0.3	.60H
0.5	.56H
0.7	.48H

m	$P_H(\frac{H}{Q_P})$	R
0.2	.78	.59H
0.4	.78	.59H
0.6	.45	.48H

FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

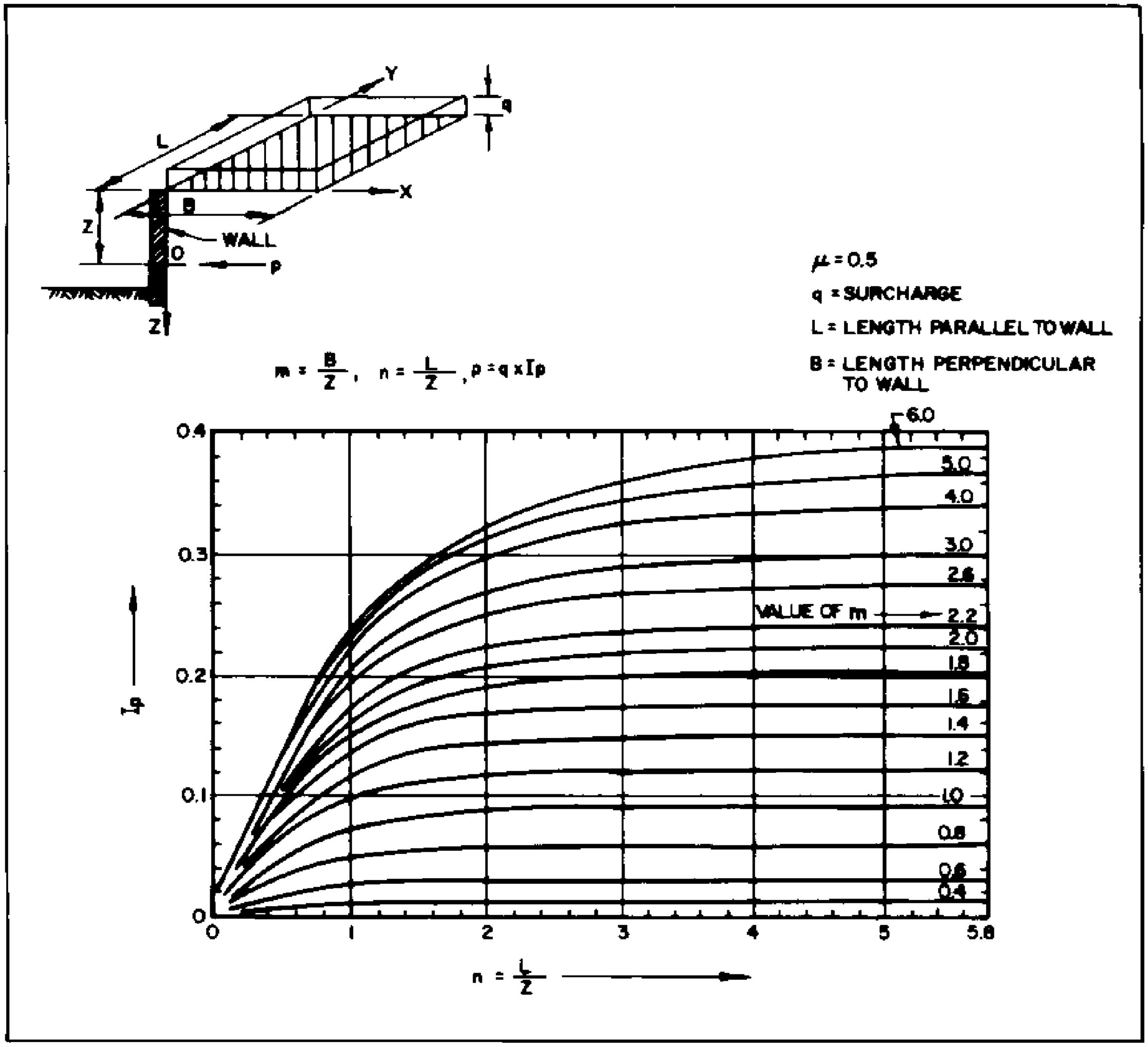


FIGURE 12
Lateral Pressure on an Unyielding Wall due to
Uniform Rectangular Surface Load

e. Restrained Walls. If a wall is prevented from even slight movement, then the earth remains at or near the value of at-rest conditions. The coefficient of earth pressure at-rest, K+o,, for normally consolidated cohesive or granular soils is approximately:

K+o, = 1-sin [theta]'

where: [theta]' = effective friction angle

Thus for [theta]' = 300, K+o, = 0.5.

For over-consolidated soils and compacted soils the range of K+o, may be on the order of 1.0. In cohesionless soils, full at-rest pressure will occur only with the most rigidly supported wall. In highly plastic clays, soil may creep, and if wall movement is prevented, at-rest conditions may redevelop even after active pressures are established.

f. Basement and Other Below Grade Walls. Pressure on walls below grade may be computed based on restraining conditions that prevail, type of backfill, and the amount of compaction.

6. EFFECT OF CONSTRUCTION PROCEDURES.

a. Staged Construction. As earth pressures are influenced by wall movement, it is important to consider each stage of construction, especially with regard to brace placement and its effects.

b. Compaction. Compaction of backfill in a confined wedge behind the wall tends to increase horizontal pressures beyond those represented by active or at-rest values. For guidance on horizontal pressure computations associated with the compaction of granular soil, see Figure 13 (after Reference 7, Retaining Wall Performance During Backfilling, by Ingold).

Clays and other fine-grained soils, as well as granular soils, with considerable amount of clay and silt (>/=15%) are not normally used as backfill material. Where they must be used, the earth pressure should be calculated on the basis of "at-rest" conditions or higher pressure with due consideration to potential poor drainage conditions, swelling, and frost action.

c. Hydraulic Fills. Active pressure coefficients for loose hydraulic fill materials range from about 0.35 for clean sands to 0.50 for silty fine sands. Place hydraulic fill by procedures which permit runoff of wash water and prevent building up large hydrostatic pressures. For further guidance see discussion on dredging in DM-7.3, Chapter 3.

7. EARTHQUAKE LOADING. The pressure during earthquake loading can be computed by the Coulomb theory with the additional forces resulting from ground acceleration. For further guidance on the subject see Reference 8, Design of Earth Retaining Structures for Dynamic Loads, by Seed and Whitman. A synopsis of some material from this Reference follows:

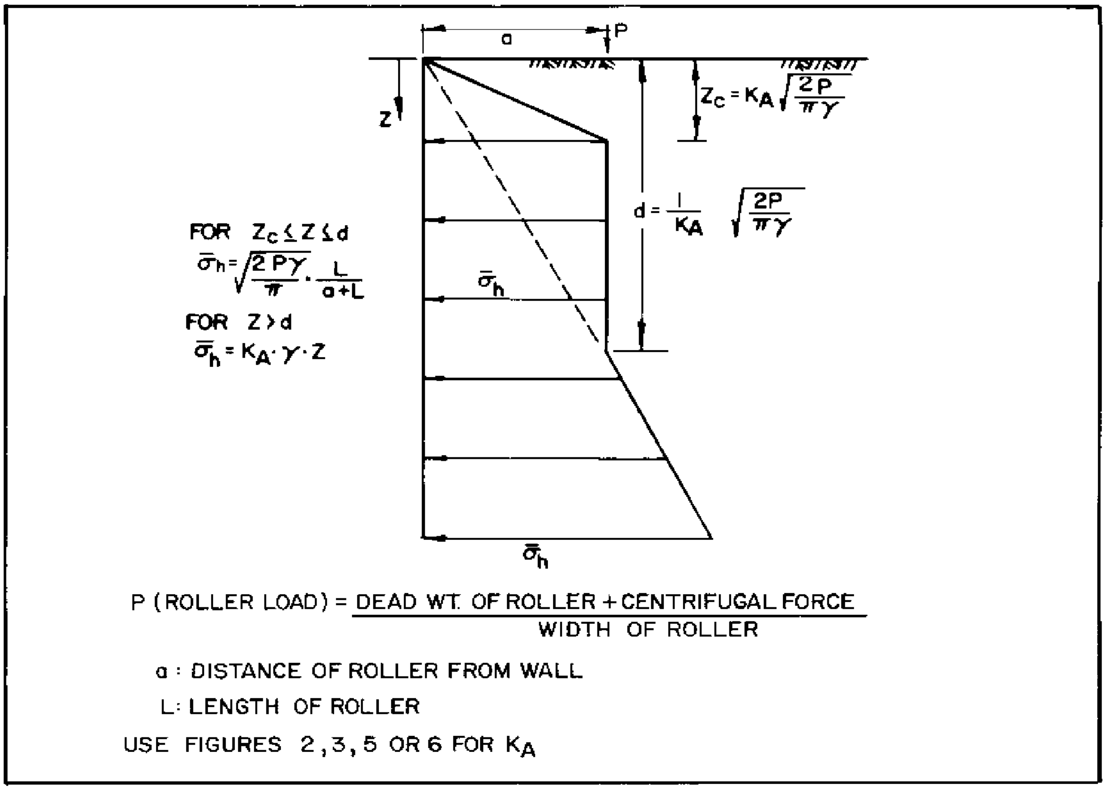


FIGURE 13
Horizontal Pressure on Walls from Compaction Effort

(1) A simple procedure for determining the lateral force due to an earthquake is to compute the initial static pressure and add to it the increase in pressure from ground motion. For a vertical wall, with horizontal backfill slope, and [theta] of 35 deg., (which may be assumed for most practical cases involving granular fill), the earth pressure coefficient for dynamic increase in lateral force can be approximated as 3/4 k+h,, k+h, being the horizontal acceleration in g's. The combined effect of static and dynamic force is:

P+AE, = 1/2 [gamma] H.2- K+A, + 3/8 [gamma] H.2-k+h,

Assume the dynamic lateral force P+E, = 3/8 [gamma].2-k+h, acts at 0.6 H above the wall base. Effect of liquefaction is considered in DM-7.3, Chapter 1.

(2) For other soil and wall properties, the combined resultant active force:

$$P_{AE} = 1/2 \ H^2 . K_A \ (\beta^* , \theta^*)(1-k_v) \ F$$

where:

$$\beta^* = \beta + \psi = \text{modified slope of backfill}$$

$$\theta^* = \theta + \psi = \text{modified slope of wall back}$$

$$\psi = \tan^{-1} \frac{k_h}{1-k_v}$$

$$F = \frac{\cos^2\theta^*}{\cos\psi \cos^2\theta}$$

k_v = vertical ground acceleration in g's.

For modified slope [beta]* and [theta]*, obtain KA([beta]*, [theta]*) from the applicable figures 3 through 8. Determine F from Figure 14. Dynamic pressure increment [W-DELTA]P+E, can be obtained by subtracting P+A, (also to be determined from Figures 3, 7, or 8 for given [beta] and [theta] values) from P+AE,. The resultant force will vary in its location depending on wall movement, ground acceleration, and wall batter. For practical purposes it may be applied at 0.6 H above the base.

(3) Unless the wall moves or rotates sufficiently, pressures greater than active case will exist and the actual lateral pressures may be as large as three times the value derived from Figure 14. In such situations, detailed analysis using numerical techniques may be desirable.

(4) Under the combined effect of static and earthquake load a factor of safety between 1.1 and 1.2 is acceptable.

(5) In cases where soil is below water, add the hydrodynamic pressure computed based on:

(p+w,)z = 1.5 k+h, [gamma]+w, (h [multiplied by] z).1/2-

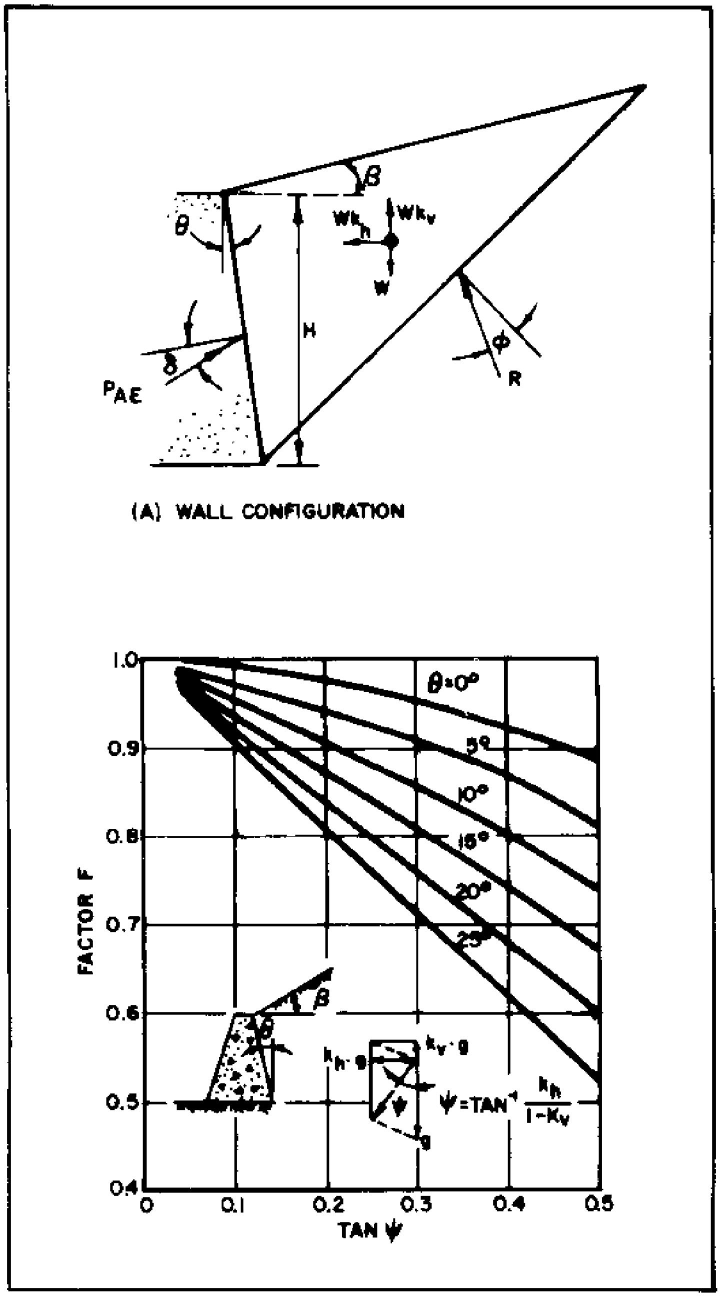
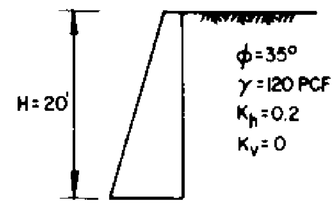
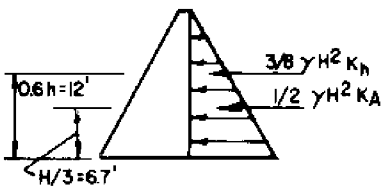


FIGURE 14(a)
Values of F for Determination of Dynamic Lateral Pressure Coefficients

EXAMPLES:

CASE 1 - VERTICAL WALL WITH HORIZONTAL BACKFILL





COMBINED EFFECT OF STATIC AND DYNAMIC FORCE.

$P_{AE} = F_1 + F_2$

$K_A = 0.27$ (FROM FIGURE 2 FOR $\phi = 35°$)

$F_1 = 1/2\ \gamma H^2\ K_A =$

$1/2\ (120)(20)^2(0.27) = 6480$ LB

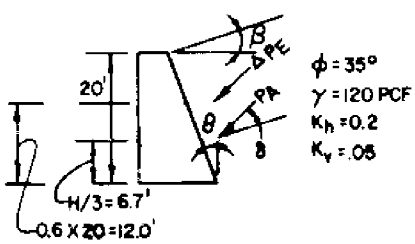
RESULTANT ACTING AT A DISTANCE OF H/3 = 6.7' FROM BASE OF WALL

$F_2 = 3/8\ \gamma H^2\ K_h =$

$3/8\ (120)(20)^2\ (0.2) = 3600$ LB.

ACTING AT 12 FT. (0.6 H) FROM BASE OF WALL

CASE 2 - SLOPING WALL WITH SLOPING BACKFILL



$\psi = \tan^{-1} \dfrac{0.2}{1-0.05} = 12°$

$\tan \psi = 0.21$

$\theta = 10°$

$\beta = 15°$

F = 0.9 (FROM FIGURE 14a)

ASSUME A SMOOTH WALL, $\delta = 0$

$\theta^* = \theta + \psi = 10 + 12 = 22°$

$\beta^* = \beta + \psi = 15 + 12 = 27°$

FROM THE EQUATION IN FIGURE 8 $K_A(\beta^*, \theta^*) = \dfrac{\cos^2(35-22)}{\cos^2 22 \cos 22 \left[1 + \sqrt{\dfrac{\sin 35 \sin(35-27)}{\cos 22 \cos(22-27)}}\right]^2} = 0.71$

$K_A(\beta, \theta) = 0.41$, $P_A = 1/2 \times (120) \times (20)^2 \times 0.41 = 9840$ LB.

$P_{AE} = 1/2\ \gamma H^2\ K_A (1 - K_V)\ F$

$= 1/2\ (120)(20)^2(0.71)(1-0.05)(0.9) = 14569$ LB.

$\Delta P_E = 14569 - 9840 = 4729$ LB.

FIGURE 14(b)
Example Calculations for Dynamic Loading on Walls

where: P_w = hydrodynamic pressure at depth z below water surface

γ_w = unit weight of water

h = depth of water

z = depth below the water surface

(6) Add the other inertia effect of the structure itself for calculating the required structural strength. An optimum design is to select the thinnest section with the largest bending and shear resistance (i.e. most flexible).

(7) When applying this earthquake loading analysis to existing earth retaining structures, particularly where high groundwater levels exist, it may be found that resulting safety factor is less than 1.1. In such cases, proposed corrective measures must be submitted to NAVFAC HQ for review and approval.

8. FROST ACTION. Lateral forces due to frost action are difficult to predict and may achieve high values.

Backfill materials such as silts and clayey silts (CL, MH, ML, OL) are frost susceptible, and will exert excessive pressure on wall if proper precautions are not taken to curb frost. Swelling pressures may be exerted by clays of high plasticity (CH). Under these conditions, design for active pressures is inadequate, even for yielding walls, as resulting wall movement is likely to be excessive and continuous. Structures usually are not designed to withstand frost generated stresses. Instead, provisions should be made so that frost related stresses will not develop or be kept to a minimum. Use of one or more of the following may be necessary:

(i) Permanently isolate the backfill from sources of water either by providing a very permeable drain or a very impermeable barrier.

(ii) Provide previous backfill and weep holes. (See DM-7.1, Chapter 6 for the illustration on complete drainage and prevention of frost thrust.)

(iii) Provide impermeable soil layer near the soil surface, and grade to drain surface water away from the wall.

9. SWELLING ACTION. Expansion of clay soils can cause very high pressures on the back of a retaining structure. Clay backfills should be avoided whenever possible. Swelling pressures may be evaluated based on laboratory tests and wall designed to withstand swelling pressures. Providing granular non-expansive filter between the clay fill and back of wall diminishes swelling pressures and significantly limits access to moisture. Guidance on soil stabilization methods for control of heave are given in DM-7.3, Chapter 3. Complete drainage (see DM-7.1, Chapter 6) is one of the techniques to control heave.

10. SELECTION OF STRENGTH PARAMETERS. The choice of strength parameters is governed by the soil permeability characteristics, boundary drainage and loading conditions, and time.

a. Saturated Cohesive Soils. For saturated cohesive soils of low permeability, where sufficient time is not available for complete drainage, use undrained shear strength, and total stress for earth pressure computations. Such condition will exist during, and immediately after completion of construction.

b. Coarse-grained Soils. In coarse-grained soils such as sand, which have high permeability, use effective stress strength parameter [theta]', for earth pressure computations. Also, where sufficient time is available for the dissipation of pore pressure in less than pervious soil, use effective stress strength parameters c' and [theta]'. In this case, pore pressure is hydrostatic and can be estimated fairly accurately.

In soils such as silt and clayey sand, where partial drainage occurs during the time of construction, perform analysis for limiting conditions, i.e. effective stress with [theta]' only, total stress with c, and design for the worst case.

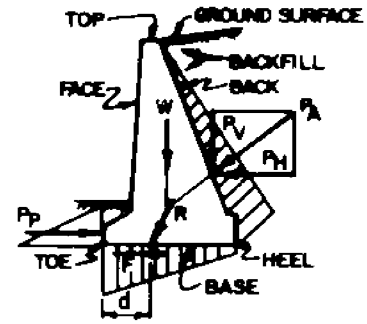
Section 3. RIGID RETAINING WALLS

1. GENERAL CRITERIA. Rigid retaining walls are those that develop their lateral resistance primarily from their own weight. Examples of rigid structures are concrete gravity walls, thick concrete slurry walls, gabion walls, and some reinforced earth walls reinforced for limited movements. Theoretical wall pressures are discussed in Section 2. Requirements for resistance against overturning and sliding of four principal wall types are given in Figure 15. Evaluate overall stability against deep foundation failure. (See DM-7.1, Chapter 7.) Determine allowable bearing pressures on the base of the wall (see Chapter 4).

a. Sliding Stability. Place the base at least 3 ft below ground surface in front of the wall and below depth of frost action, zone of seasonal volume change, and depth of scour. Sliding stability must be adequate without including passive pressure at the toe. If insufficient sliding resistance is available, increase base width, provide pile foundation or, lower base of wall and consider passive resistance below frost depth. If the wall is supported by rock or very stiff clay, a key may be installed below the foundation to provide additional resistance to sliding (see Figure 15).

b. Settlement and Overturning. For walls on relatively incompressible foundations, apply overturning criteria of Figure 15. If foundation is compressible, compute settlement by methods of DM-7.1, Chapter 5 and estimate tilt of rigid wall from the settlement. If the consequent tilt will exceed acceptable limits, proportion the wall to keep the resultant force at the middle third of base. If a wall settles such that the resulting movement forces it into the soil which it supports, then the lateral pressure on the active side increases substantially.

c. Overall Stability. Where retaining walls are underlain by weak soils, the overall stability of the soil mass containing the retaining wall should be checked with respect to the most critical surface of sliding (see DM-7.1, Chapter 7). A minimum factor of safety of 2.0 is desirable.

<table>
<tr><th>TYPE OF WALL</th><th>LOAD DIAGRAM</th><th>DESIGN FACTORS</th></tr>
<tr><td>GRAVITY</td><td>
</td><td rowspan="4">

LOCATION OF RESULTANT

MOMENTS ABOUT TOE:

$$d = \frac{Wa + P_V e - P_H b}{W + P_V}$$

ASSUMING $P_P = 0$

OVERTURNING

MOMENTS ABOUT TOE:

$$F_S = \frac{Wa}{P_H b - P_V e} \geq 1.5$$

IGNORE OVERTURNING IF R IS WITHIN MIDDLE THIRD (SOIL), MIDDLE HALF (ROCK).

CHECK R AT DIFFERENT HORIZONTAL PLANES FOR GRAVITY WALLS.

RESISTANCE AGAINST SLIDING

$$F_S = \frac{(W+P_V)\,\mathrm{TAN}\,\delta + C_a B}{P_H} \geq 1.5$$

$$F_S = \frac{(W+P_V)\,\mathrm{TAN}\,\delta + C_a B + P_P}{P_H} \geq 2.0$$

$F = (W+P_V)\,\mathrm{TAN}\,\delta + C_a B$

FOR COEFFICIENTS OF FRICTION BETWEEN BASE AND SOIL SEE TABLE-1.

C_a = ADHESION BETWEEN SOIL AND BASE

TAN δ = FRICTION FACTOR BETWEEN SOIL AND BASE

W = INCLUDES WEIGHT OF WALL AND SOIL IN FRONT FOR GRAVITY AND SEMIGRAVITY WALLS. INCLUDES WEIGHT OF WALL AND SOIL ABOVE FOOTING, FOR CANTILEVER AND COUNTERFORT WALLS.

CONTACT PRESSURE ON FOUNDATION

FOR ALLOWABLE BEARING PRESSURE FOR INCLINED LOAD ON STRIP FOUNDATION, SEE CHAPTER 4.

FOR ANALYSIS OF PILE LOADS BENEATH STRIP FOUNDATION, SEE CHAPTER 7.

OVERALL STABILITY

FOR ANALYSIS OF OVERALL STABILITY, SEE DM-7.1, CHAPTER 7.

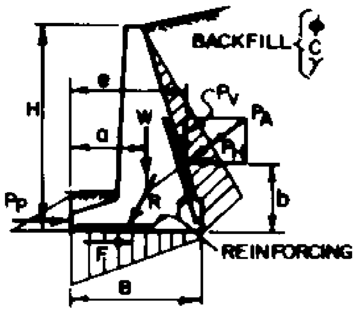
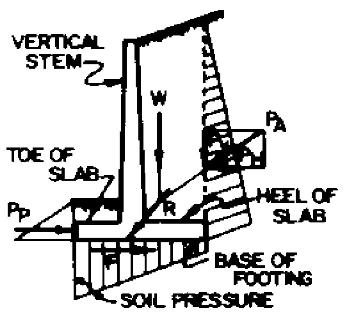
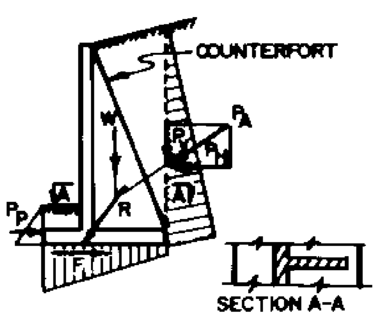
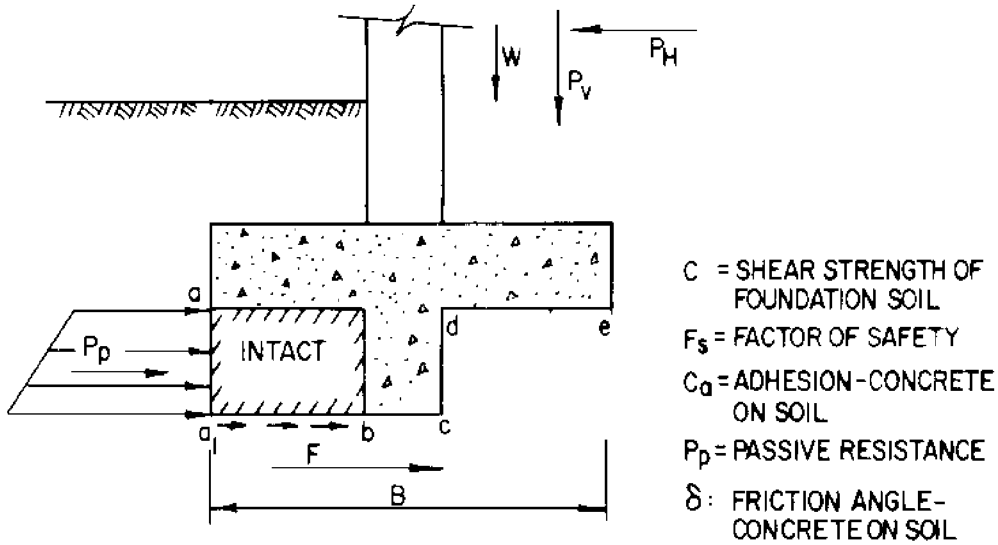
</td></tr>
<tr><td>SEMIGRAVITY</td><td>
</td></tr>
<tr><td>CANTILEVER</td><td>
</td></tr>
<tr><td>COUNTERFORT</td><td>
</td></tr>
</table>

FIGURE 15
Design Criteria for Rigid Retaining Walls



RESISTANCE AGAINST SLIDING ON KEYED FOUNDATIONS

COHESIVE SOILS $F = (W+P_V)\ TAN\ \delta + C_a(B-\overline{a_1b}) + C\ (\overline{a_1b}) + P_p$

GRANULAR SOILS $F = (W+P_V)\ TAN\ \delta + P_p$

$$F_s = \frac{F}{P_H}$$

FIGURE 15 (continued)
Design Criteria for Rigid Retaining Walls

d. Drainage 1. Positive drainage of backfill is desirable. (see DM-7.1, Chapter 6 for drainage design.) As a minimum, provide weep holes with pockets of coarse-grained material at the back of the wall. An impervious surface layer should cover the backfill, and a gutter should be provided for collecting runoff.

2. LOW WALLS. It has been the practice of the Naval Facilities Engineering Command to consider walls less than 12 feet in height "low walls." For these, knowledge of soil properties could be adequate for design, and detailed testing and elaborate pressure computations may not be justified economically.

a. Equivalent Fluid Pressures. Use equivalent fluid pressures of Figure 16 (Reference 9, Soil Mechanics in Engineering Practice, by Terzaghi and Peck) for straight slope backfill and of Figure 17 (Reference 9) for broken slope backfill. Include dead load surcharge as an equivalent weight of backfill. For resultant force of line load surcharge, see bottom left panel of Figure 11. If a wall rests on a compressible foundation and moves downward with respect to the backfill, increase pressures by 50 percent.

b. Drainage. The equivalent fluid pressures include effects of seepage and time conditioned changes in the backfill. However, provisions should be made to prevent accumulation of water behind the wall. As a minimum, provide weep holes for drainage. Cover backfill of soil types 2 and 3 (Figure 16) with a surface layer of impervious soil.

Section 4. DESIGN OF FLEXIBLE WALLS

1. ANCHORED BULKHEADS. Anchored bulkheads are formed of flexible sheeting restrained by tieback and by penetration of sheeting below dredge line. See Figure 18 for design procedures for three common penetration conditions.

a. Wall Pressures. Compute active and passive pressures using the appropriate Figures 2 through 7. Determine required depth of penetration of sheeting and anchor pull from these pressures. See Figure 18 for guidance.

b. Wall Movements. Active pressures are redistributed on the wall by deflection, moving away from the position of maximum moment. Reduce the computed maximum moment to allow for flexibility of sheeting. Moment reduction is a function of the wall flexibility number. See Figure 19 (Reference 10, Anchored Sheet Pile Walls, by Rowe). Select sheeting size by successive approximations so that sheeting stiffness is compatible with reduced design moment.

c. Drainage. Include the effect of probable maximum differential head in computing wall pressures. Where practicable, provide weep holes or special drainage at a level above mean water to limit differential water pressures.

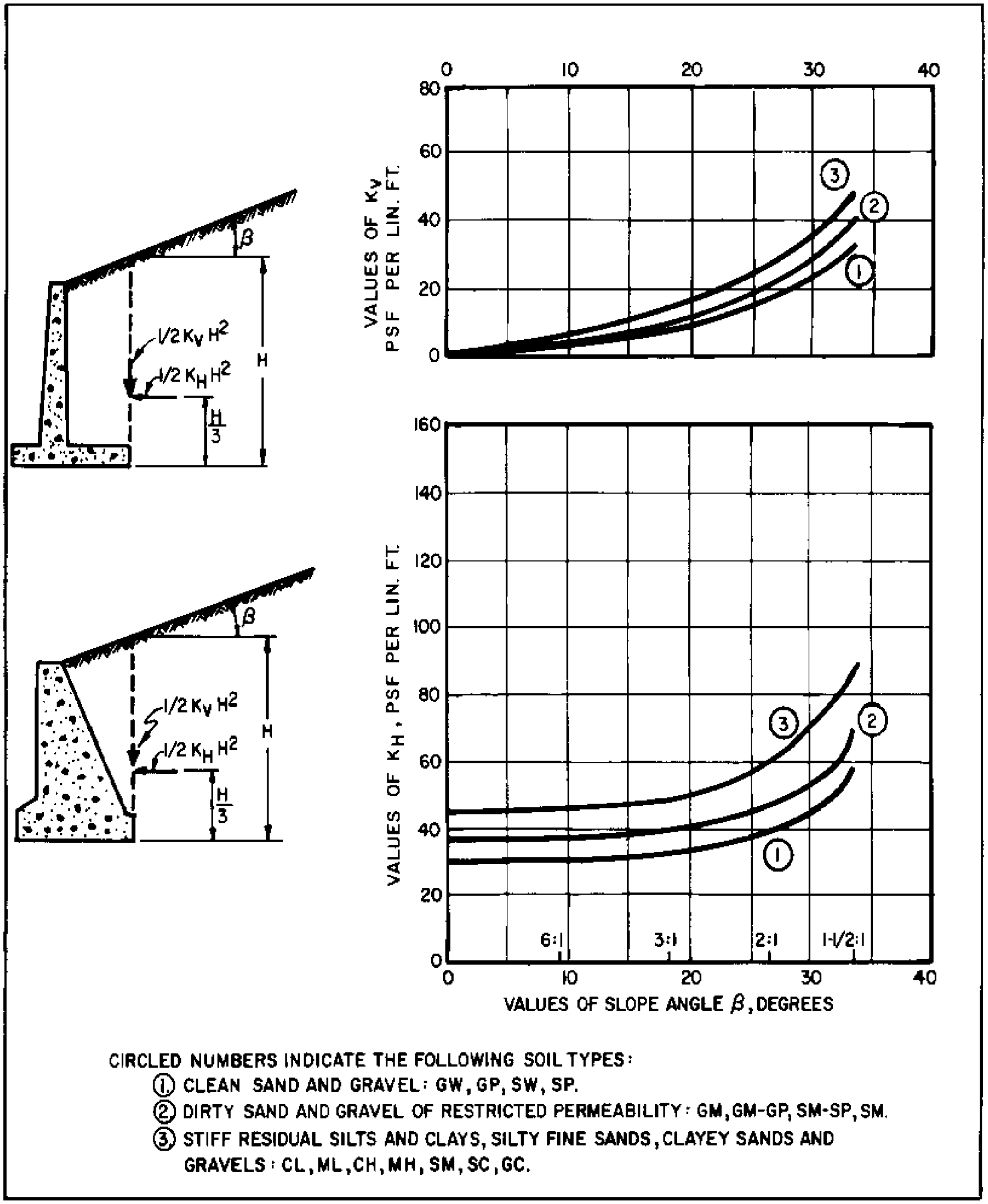
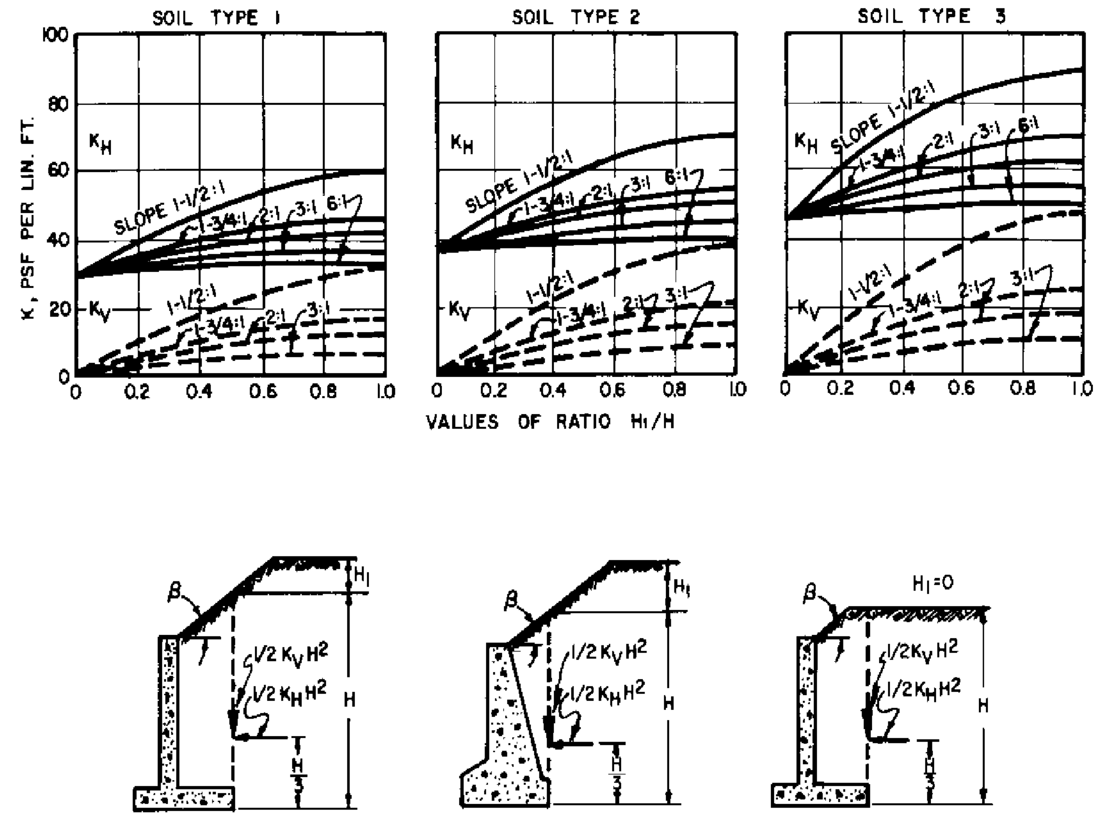
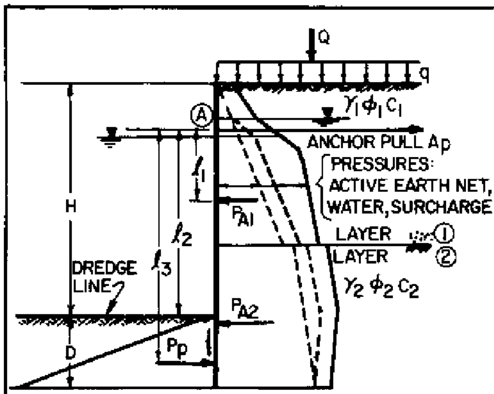


FIGURE 16
Design Loads for Low Retaining Walls (Straight Slope Backfill)



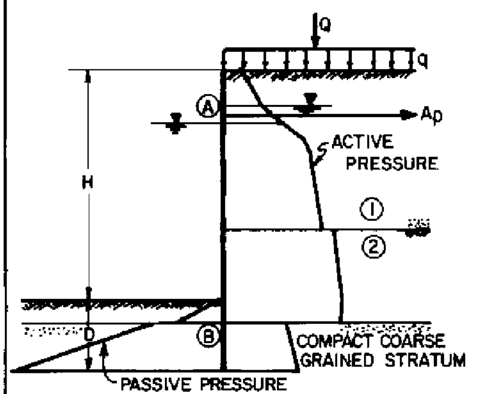
FOR DESCRIPTION OF SOIL TYPE SEE FIGURE 16

FIGURE 17
Design Loads for Low Retaining Walls (Broken Slope Backfill)



FREE EARTH SUPPORT - GENERAL CASE

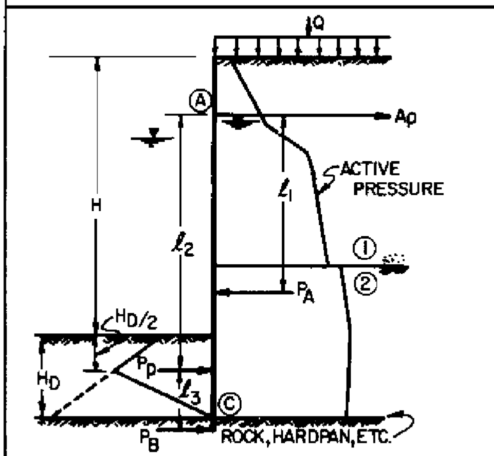
1. COMPUTE PRESSURES BY METHODS OF FIGURES 2 TO 7 PASSIVE PRESSURES FOR CLEAN COARSE GRAIN SOILS INCLUDE WALL FRICTION (δ). TABLE I. FOR ACTIVE OR PASSIVE PRESSURES IN ALL OTHER SOIL TYPES, IGNORE WALL FRICTION.
2. DEPTH OF PENETRATION REQUIRED: TAKE MOMENTS ABOUT POINT Ⓐ AND SOLVE FOR D: $P_{A1}\ell_1 + P_{A2}\ell_2 = \frac{P_p}{F_S}\ell_3$
   F_S = 2 TO 3 FOR COARSE GRAINED SOILS
   F_S = 1.5 TO 2 FOR FINE GRAINED SOILS
3. ANCHOR PULL: $A_p = [P_{A1} + P_{A2} - P_p/F_S]\,d$, d = ANCHOR SPACING
4. MAXIMUM BENDING MOMENT (M_{MAX}) IN SHEETING COMPUTED BY THE FREE EARTH SUPPORT METHOD AND APPLYING P_{A1}, P_{A2}, P_p/F_S AND Ap. FOR SHEETING IN SAND APPLY MOMENT REDUCTION FOR FLEXIBILITY OF FIGURE 19.
5. INCRESE PENETRATION COMPUTED (D) BY 20% TO ALLOW FOR DREDGING, SCOUR, ETC.



PENETRATION IN COMPACT COARSE GRAINED STRATUM

DESIGN STEPS 1, 2, AND 3 SAME AS ABOVE EARTH SUPPORT.

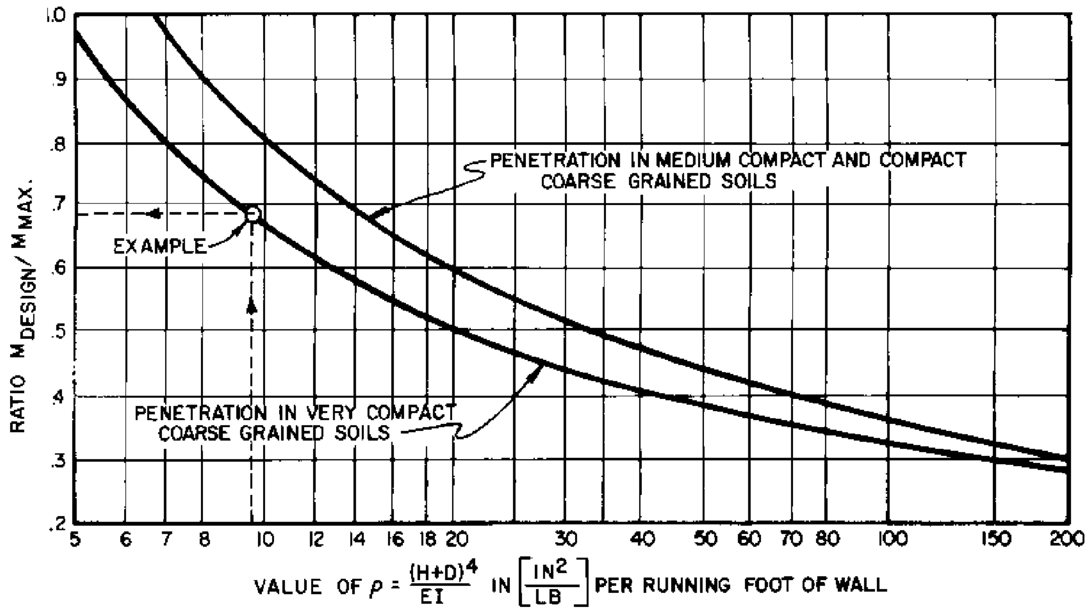
4. COMPUTE MAXIMUM BENDING MOMENT (M_{MAX}) IN SHEETING BY FREE EARTH SUPPORT METHOD APPLYING P_A, P_p/F_S AND Ap.
5. COMPUTE ρ ACCORDING TO FIGURE 19. IF $\rho \geq 20$, M_{DESIGN} IS COMPUTED FOR THE SPAN ⒶⒷ ASSUMING SIMPLE SUPPORT AT POINT Ⓑ
   IF $\rho < 20$ OBTAIN MOMENT REDUCTION FOR FLEXIBILITY FROM FIGURE 19.
6. INCREASE PENETRATION COMPUTED (D) BY 20% TO ALLOW FOR DREDGING, SCOUR, ETC.

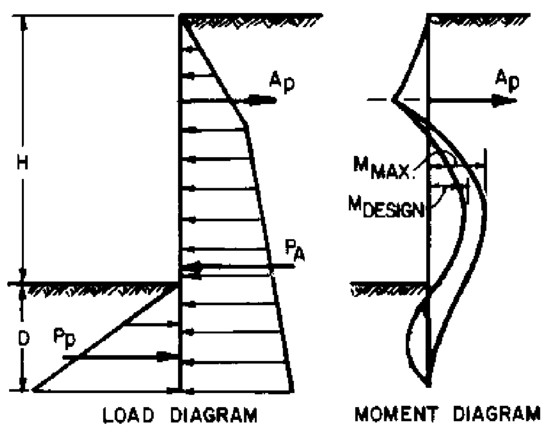


PENETRATION TO TOP OF HARD UNYIELDING STRATUM

1. COMPUTE PRESSURES AS ABOVE. EXCEPT THAT PASSIVE PRESSURE DECREASES TO ZERO AT TOP OF HARD STRATUM.
2. PENETRATION IN HARD STRATUM: TAKE MOMENTS ABOUT POINT Ⓐ AND SOLVE FOR P_B:
   $$P_A\ell_1 - \frac{P_p}{F_S}\ell_2 = \frac{P_B}{F_S}(\ell_2 + \ell_3)$$
   ESTIMATE IF REACTION P_B CAN BE PROVIDED BY SHALLOW PENETRATION IN HARD STRATUM.
3. ANCHOR PULL: $A_p = \left[P_A - \frac{P_p}{F_S} - \frac{P_B}{F_S}\right] d$
4. MAXIMUM BENDING MOMENT IN SHEETING COMPUTED BY APPLYING P_A, P_p AND A_p TO SPAN Ⓐ-Ⓒ ASSUMING SIMPLE SUPPORT AT Ⓒ. NO REDUCTION FOR FLEXIBILITY.

FIGURE 18
Design Criteria for Anchored Bulkhead (Free Earth Support)





<u>EXAMPLE</u>: PENETRATION IN VERY COMPACT SAND

M_{MAX} =950,000 IN. LB/FT.

H = 33 FT, D = 15 FT.

f_S = 25,000 PSI, E=30,000,000 PSI

TRY ZP 32, I = 385.7 $IN.^4$, S = 38.3 $IN.^3$

$$\rho = \frac{(33+15)^4 \times 12^4}{30{,}000{,}000 \times 385.7} = 9.5 \ \frac{IN.^2}{LB.}$$

$$\frac{M_{DESIGN}}{M_{MAX.}} = 0.68, \quad M_{DESIGN} = 645{,}000 \text{ IN. LB/FT}$$

$$f_S = \frac{M}{S} = \frac{645{,}000}{38.3} = 16{,}800 \text{ PSI}$$

16,800 < 25,000 PSI

TRY A SMALLER SECTION.

<u>LEGEND</u>

M_{MAX} = MAXIMUM POSITIVE MOMENT IN SHEETING COMPUTED BY METHODS OF FIGURE 18.

M_{DESIGN} = MAXIMUM POSITIVE MOMENT FOR DESIGN OF SHEETING.

ρ = FLEXIBILITY NUMBER = $\frac{(H+D)^4}{EI}$, E = SHEETING MODULUS OF ELASTICITY, PSI
I = SHEETING MOMENT OF INERTIA, $IN.^4$ PER RUNNING FOOT OF WALL.

<u>NOTES</u>

1. M_{DESIGN} IS OBTAINED BY SUCCESSIVE TRIALS OF SHEETING SIZE UNTIL MAX. BENDING STRESS IN SHEETING EQUALS ALLOWABLE BENDING STRESS.
2. NO REDUCTION IN $M_{MAX.}$ IS PERMITTED FOR PENETRATION IN FINE GRAINED SOILS OR LOOSE OR VERY LOOSE COARSE GRAINED SOILS.
3. FLEXIBILITY NUMBER IS COMPUTED ON THE BASIS OF LUBRICATED INTERLOCKS.

FIGURE 19
Reduction in Bending Moments in Anchored Bulkhead from Wall Flexibility

d. Anchorage System. Most of the difficulties with anchored bulkheads are caused by their anchorage. A tieback may be carried to a buried deadman anchorage, to pile anchorage, parallel wall anchorage, or it may be a drilled and grouted anchor (see DM-7.3, Chapter 3). See Figure 20 for criteria for design of deadman anchorage. If a deadman must be positioned close to a wall, anchorage resistance is decreased and an additional passive reaction is required for stability at the wall base. Protect tie rods by wrapping, painting, or encasement to resist corrosion. Where backfill will settle significantly or unevenly, to avoid loading by overburden, enclose tie rod in a rigid tube, providing vertical support if needed to eliminate sag.

e. Example of Computation. See Figure 21 for example of analysis of anchored bulkhead.

f. Construction Precautions. Precautions during construction are as follows:

(1) Removal of soft material, or placement of fill in the "passive" zone should precede the driving of sheet piles.

(2) Deposit backfill by working away from the wall rather than toward it to avoid trapping, a soft material adjacent to sheeting.

(3) Before anchorage is placed, sheeting is loaded as a cantilever wall, and safety during construction stages should be checked.

g. Sand Dike Backfill. When granular backfill is scarce, a sand dike may be place to form a plug across the potential failure surface of the active wedge as shown in Figure 22. Where such a dike rests on firm foundation soil, the lateral pressure on the bulkhead will be only the active pressure of the dike material. For further guidance, see Reference 11, Foundations, Retaining and Earth Structures, by Tschebotarioff.

2. CANTILEVER SHEET PILE WALLS. A cantilever wall derives support from the passive resistance below the dredge line to support the active pressure from the soil above the dredge line without an anchorage. This type of wall is suitable only for heights up to about 15 feet and can be used only in granular soils or stiff clays. See Figure 23 for a method of analysis (after Reference 12, Steel Sheet Piling Design Manual, by U.S. Steel Corporation). For cohesive soils consider no negative pressure in tension zone. Figures 24 and 25 (Reference 12) may be used for simple cases.

3. INTERNALLY BRACED FLEXIBLE WALLS. To restrain foundation or trench excavations, flexible walls can be braced laterally as the excavation proceeds. This restrains lateral movement of the soil and cause loads on the braces which exceed those expected from active earth pressure. Braces may be either long raking braces or relatively short horizontal cross braces between trench walls. Design earth pressure diagram for internally braced flexible walls are shown in Figure 26 (after Reference 6) for excavations in sand, soft clay, or stiff clay.

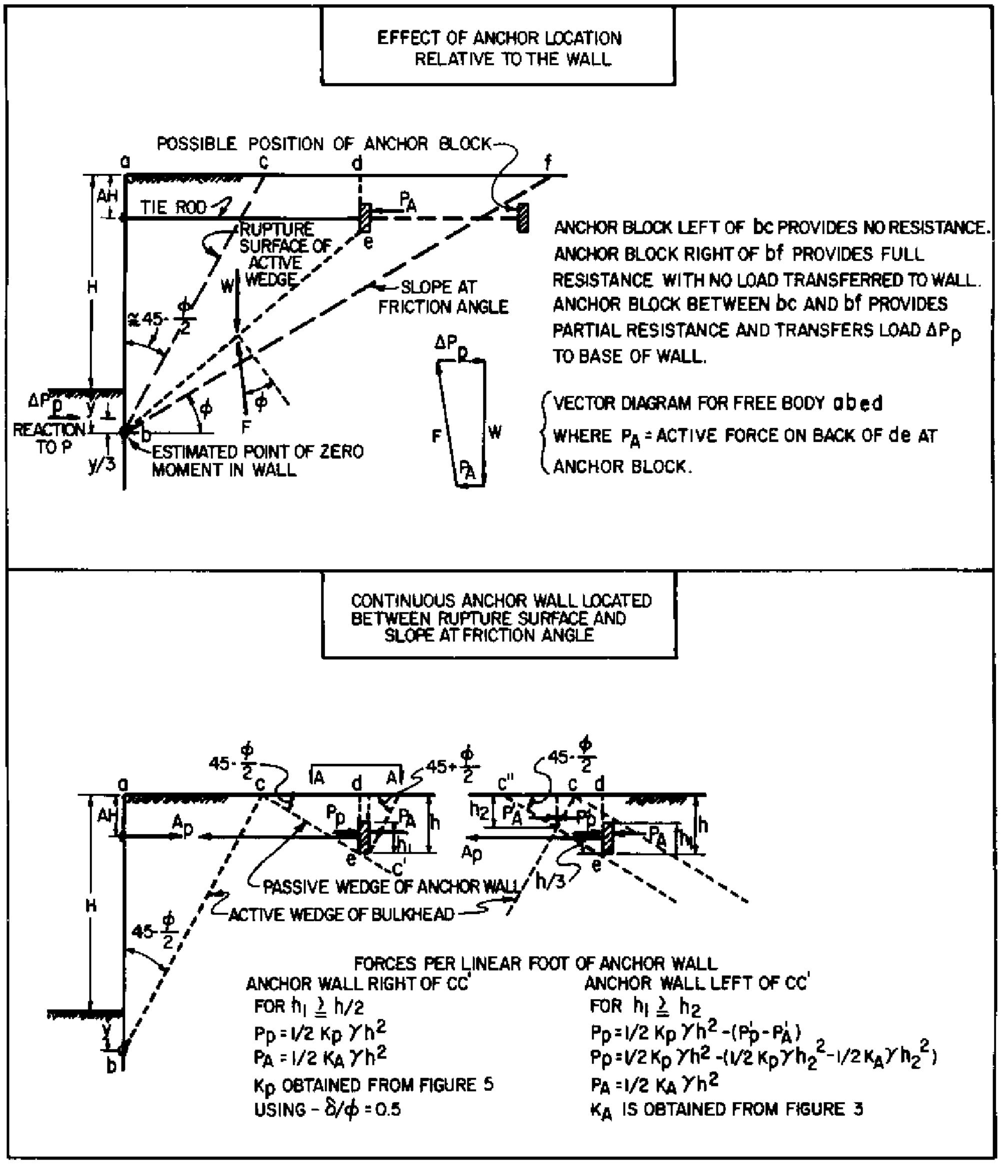
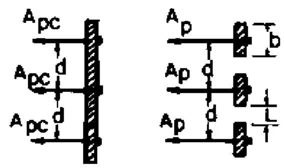


FIGURE 20
Design Criteria for Deadman Anchorage

EFFECT OF DEPTH AND SPACING OF ANCHOR BLOCKS



SECTION A-A CONTINUOUS WALL

SECTION A-A INDIVIDUAL ANCHORS

ANCHOR RESISTANCE FOR $h_1 \geq \frac{h}{2}$

1. CONTINUOUS WALL:

   ULTIMATE $A_{pc}/d = P_p - P_A$ WHERE A_{pc}/d IS ANCHOR RESISTANCE AND P_p, P_A TAKEN PER LINEAL FOOT OF WALL.

2. INDIVIDUAL ANCHORS:

   IF $d > b+h$, ULTIMATE $A_p = b(P_p - P_A) + 2P_0 \tan\phi$, WHERE P_0 = RESULTANT FORCE OF SOIL AT REST ON VERTICAL AREA cde OR c''de.

   IF $d = h+b$, A_p/d IS 70% OF A_{pc}/d FOR CONTINUOUS WALL. L FOR THIS CONDITION IS L' AND L' = h.

   IF $d < h+b$, $A_p/d = A_{pc}/d - \frac{L}{L'}(.3\,A_{pc}/d)$, $L' = h$.

ANCHOR RESISTANCE FOR $h_1 < \frac{h}{2}$

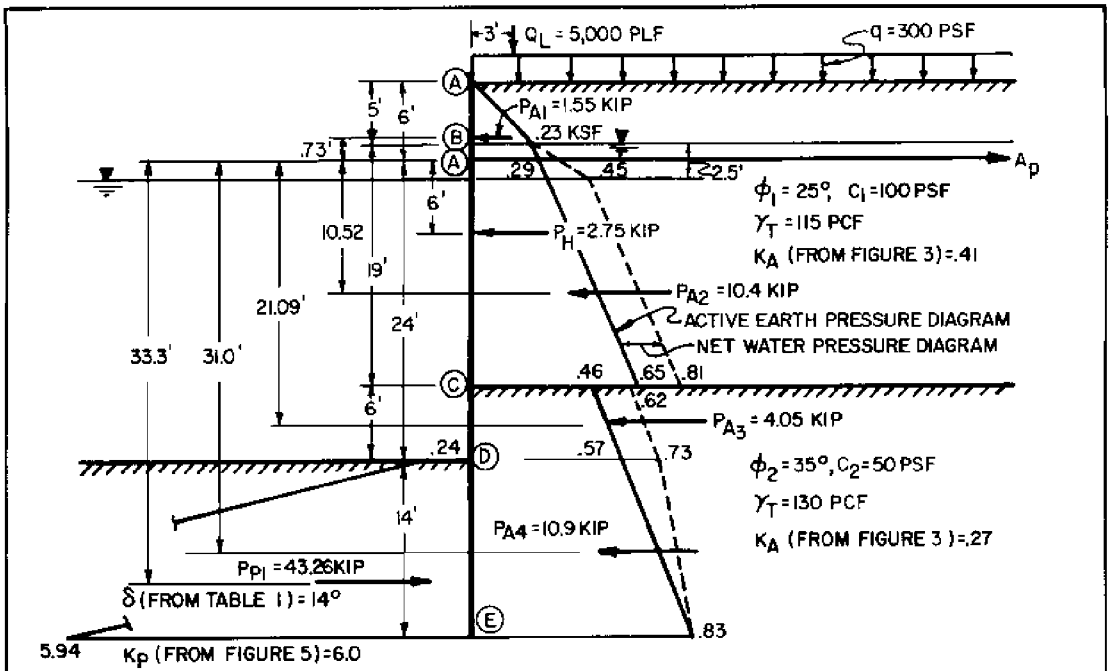
   ULTIMATE A_p/d OR A_{pc}/d EQUALS BEARING CAPACITY OF STRIP FOOTING OF WIDTH h_1 AND SURCHARGE LOAD $\gamma(h - \frac{h_1}{2})$, SEE FIGURE 1, CHAPTER 4

   USE FRICTION ANGLE ϕ': WHERE $\tan\phi' = 0.6 \tan\phi$.

GENERAL REQUIREMENTS:

1. ALLOWABLE VALUE OF A_p AND A_{pc} = ULTIMATE VALUE/2, FACTOR OF SAFETY OF 2 AGAINST FAILURE.
2. VALUES OF K_A AND K_p ARE FOR COHESIONLESS MATERIALS. IF BACKFILL HAS BOTH ϕ AND c STRENGTHS, COMPUTE ACTIVE AND PASSIVE FORCES ACCORDING TO FIGURES 7 AND 9. FINE GRAINED SOILS OF MEDIUM TO HIGH PLASTICITY SHOULD NOT BE USED AT THE ANCHORAGE.
3. SOILS WITHIN PASSIVE WEDGE OF ANCHORAGE SHALL BE COMPACTED TO NO LESS THAN 90% OF MAX. UNIT WEIGHT (ASTM D698 TEST).
4. TIE ROD IS DESIGNED FOR ALLOWABLE A_p OR A_{pc}. TIE ROD CONNECTIONS TO WALL AND ANCHORAGE ARE DESIGNED FOR 1.2 (ALLOWABLE A_p OR A_{pc}).
5. TIE ROD CONNECTION TO ANCHORAGE IS MADE AT THE LOCATION OF THE RESULTANT EARTH PRESSURES ACTING ON THE VERTICAL FACE OF THE ANCHORAGE.

FIGURE 20 (continued)
Design Criteria for Deadman Anchorage



ACTIVE EARTH PRESSURE (SEE FIGURE 2)
INCLUDING UNIFORM SURCHARGE q

$\sigma_H = \gamma Z K_A - 2C\sqrt{K_A}$

Ⓐ, $\sigma_H = .30 \times .41 - 2 \times .10\sqrt{.41} = 0$

Ⓑ, $\sigma_H = (.30 + 5 \times .115).41 - 2 \times .10\sqrt{.41} = .23$ KSF

Ⓒ, $\sigma_H = (.30 + 5 \times .115 + 19 \times .053).41 - 2 \times .10\sqrt{.41} = .65$ KSF

Ⓒ, $\sigma_H = (.30 + 5 \times .115 + 19 \times .053).27 - 2 \times .05\sqrt{.27} = .46$ KSF

Ⓓ, $\sigma_H = .46 + 6 \times .068 \times .27 = .57$ KSF

Ⓔ, $\sigma_H = .46 + (6 + 14) \times .068 \times .27 = .83$ KSF

PRESSURE OF LINE LOAD SURCHARGE
(SEE FIGURE 11)

$m = \frac{X}{H} = \frac{3}{30} = 0.1$

$P_H = 0.55\,Q_L = 0.55 \times 5 = 2.75$ KIP

LOCATION OF RESULTANT:

$R = .60H = .60 \times 30 = 18'$

NET WATER PRESSURE

$\sigma_W = \gamma_W Z = .0625 \times 2.5 = .16$ KSF

PASSIVE PRESSURE

$\sigma_H = \gamma Z K_P + 2C\sqrt{K_P}$

Ⓓ, $\sigma_H = 0 + 2 \times .05\sqrt{6.0} = .24$ KSF

Ⓔ, $\sigma_H = .068 \times 14 \times 6.0 + 2 \times .05\sqrt{6.0} = 5.94$ KSF

SAFETY FACTOR AGAINST TOE FAILURE:

TAKE MOMENTS ABOUT Ⓐ

$$F_S = \frac{\Sigma \text{ MOMENTS OF PASSIVE FORCES}}{\Sigma \text{ MOMENTS OF ACTIVE FORCES}}$$

$$= \frac{43.26 \times 33.3}{1.55 \times 0.73 + 2.75 \times 6 + 10.4 \times 10.52 + 4.05 \times 21.09 + 10.9 \times 31.0}$$

$= 2.61 > 2.5$

ANCHOR PULL

$A_P = \Sigma P_A - \Sigma P_P / F_S$

$= 1.55 + 2.75 + 10.4 + 4.05 + 10.9 - \frac{43.26}{2.5} = 12.37$ KIPS

MAXIMUM BENDING MOMENT IN SHEETING

POINT OF ZERO SHEAR:

$12.37 - 1.55 - 2.75 - .45X - .022 \times \frac{X^2}{2} = 0$

$X \cong 13.6'$ BELOW OUTSIDE WATER LEVEL

$M_{MAX} = 1.55 \times 15.7 + 12.37 \times 15.1 - 2.75 \times 9.1$

$- .45 \times \frac{13.6^2}{2} - .022 \times \frac{13.6^2}{2} \times 4.52 = 86.9$ FT-KIPS

MOMENT REDUCTION:

ASSUME: $f_S = 27{,}000$ PSI, $E = 30{,}000{,}000$ PSI

TRY ZP32, $I = 385.7\ IN^4$, $S = 38.3\ IN^3$

ρ = (FROM FIGURE 19) $= \frac{(H+D)^4}{EI}$

$\rho = \frac{(30 \times 12 + 14 \times 12)^4}{30{,}000{,}000 \times 385.7} = 6.7\ \frac{IN^2}{LB}$

$\frac{M_{DESIGN}}{M_{MAX}} = .83$; $M_{DESIGN} = .83 \times 86.9 = 72.1$ FT-KIPS

$f_S = \frac{M}{S} = \frac{72.1 \times 1000 \times 12}{38.3} = 22{,}600\ PSI < 27{,}000$

TRY A SMALLER SECTION

FIGURE 21
Example of Analysis of Anchored Bulkhead

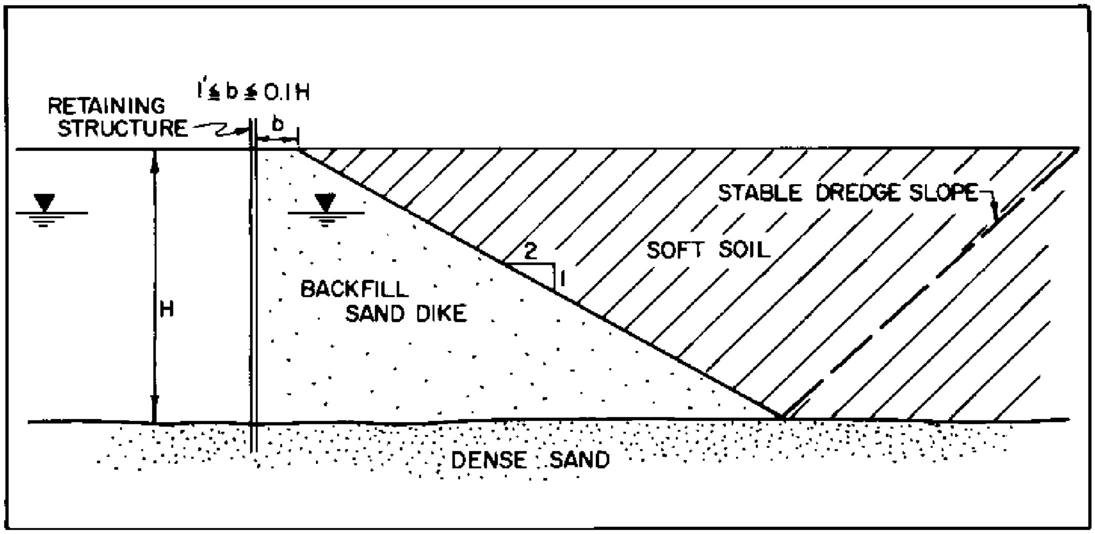
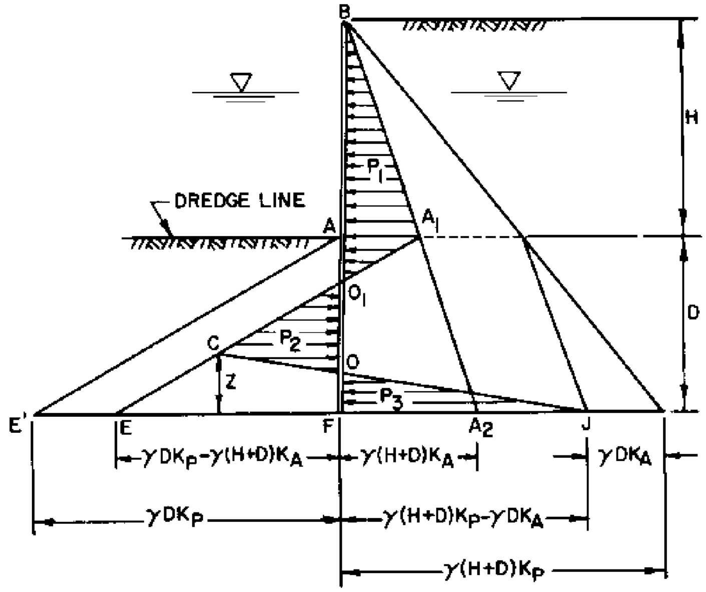


FIGURE 22
Sand Dike Scheme for Controlling Active Pressure

NOTE: WATER LEVELS CAN BE DIFFERENT ON OPPOSITE SIDES DUE TO PUMPING, TIDAL FLUCTUATIONS AND OTHER REASONS.



1. Assume a trial depth of penetration, D. This may be estimated from the following approximate correlation.

Standard Penetration Resistance, N Blows/foot	Depth of Penetration*
0 - 4	2.0H
5 - 10	1.5H
11 - 30	1.25H
31 - 50	1.0H
+50	0.75H

* H = height of piling above dredge line

2. Determine the active and passive lateral pressure using appropriate coefficients of lateral earth pressure. If the Coulomb method is used, it should be used conservatively for the passive pressure.

3. Satisfy the requirements of static equilibrium: the sum of the forces in the horizontal direction must be zero and the sum of the moments about any point must be zero. The sum of the horizontal forces may be written in terms of pressure areas:

$$\Delta(\overrightarrow{EA_1A_2}) - \Delta(\overleftarrow{FBA_2}) - \Delta(\overleftarrow{ECJ}) = 0$$

Solve the above equation for the distance, Z. For a uniform granular soil,

$$Z = \frac{K_P\ D^2 - K_A\ (H+D)^2}{(K_P - K_A)\ (H+2D)}$$

FIGURE 23
Analysis for Cantilever Wall

4. Take moments about point F. If sum of moments is other than zero readjust D and repeat calculations until sum of moments around F is zero.

5. Compute maximum moment at point of zero shear.

6. Increase D by 20% - 40% to result in approximate factory of safety of 1.5 to 2.

FIGURE 23 (continued)
Analysis for Cantilever Wall

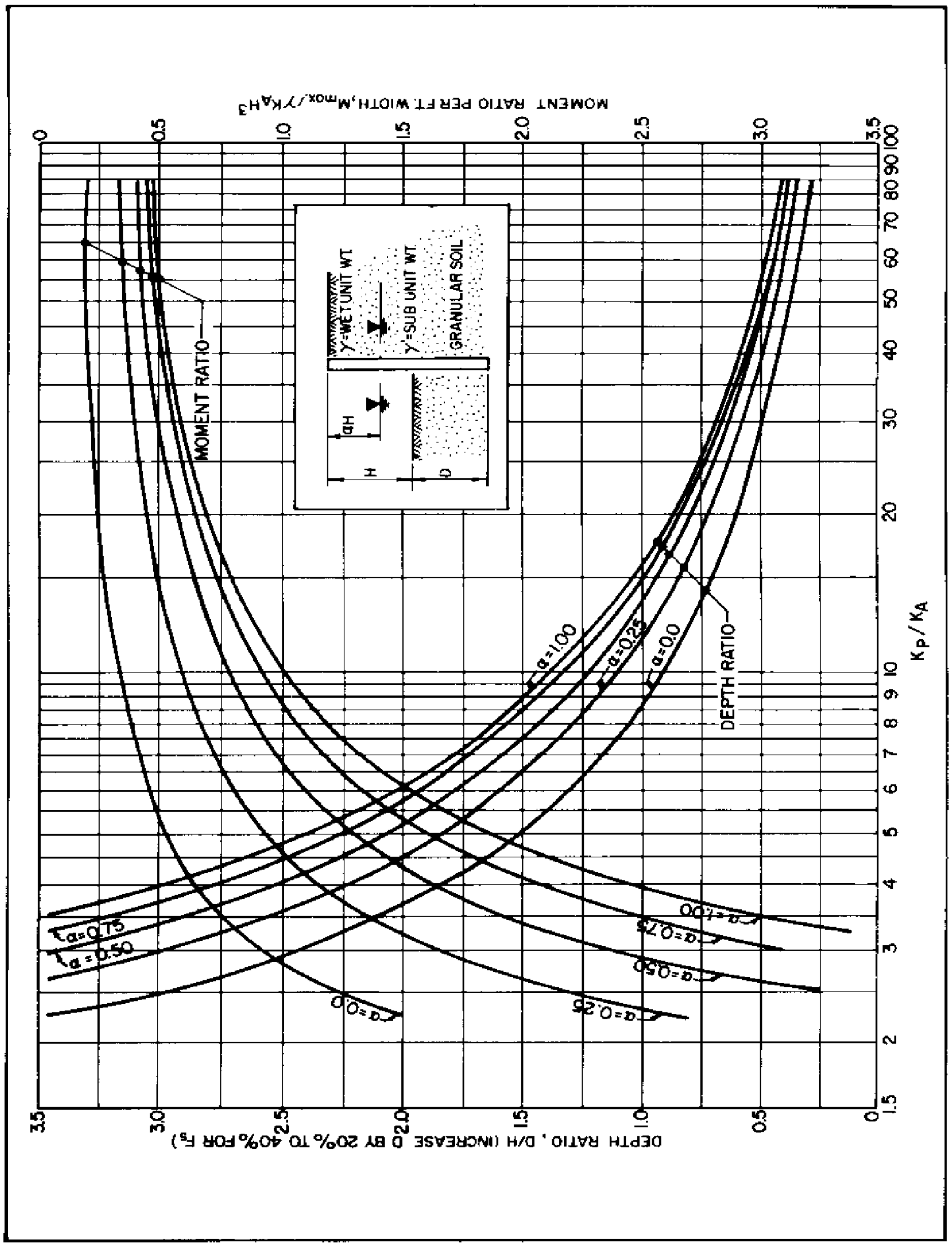


FIGURE 24
Cantilever Steel Sheet Pile Wall in Homogeneous Granular Soil

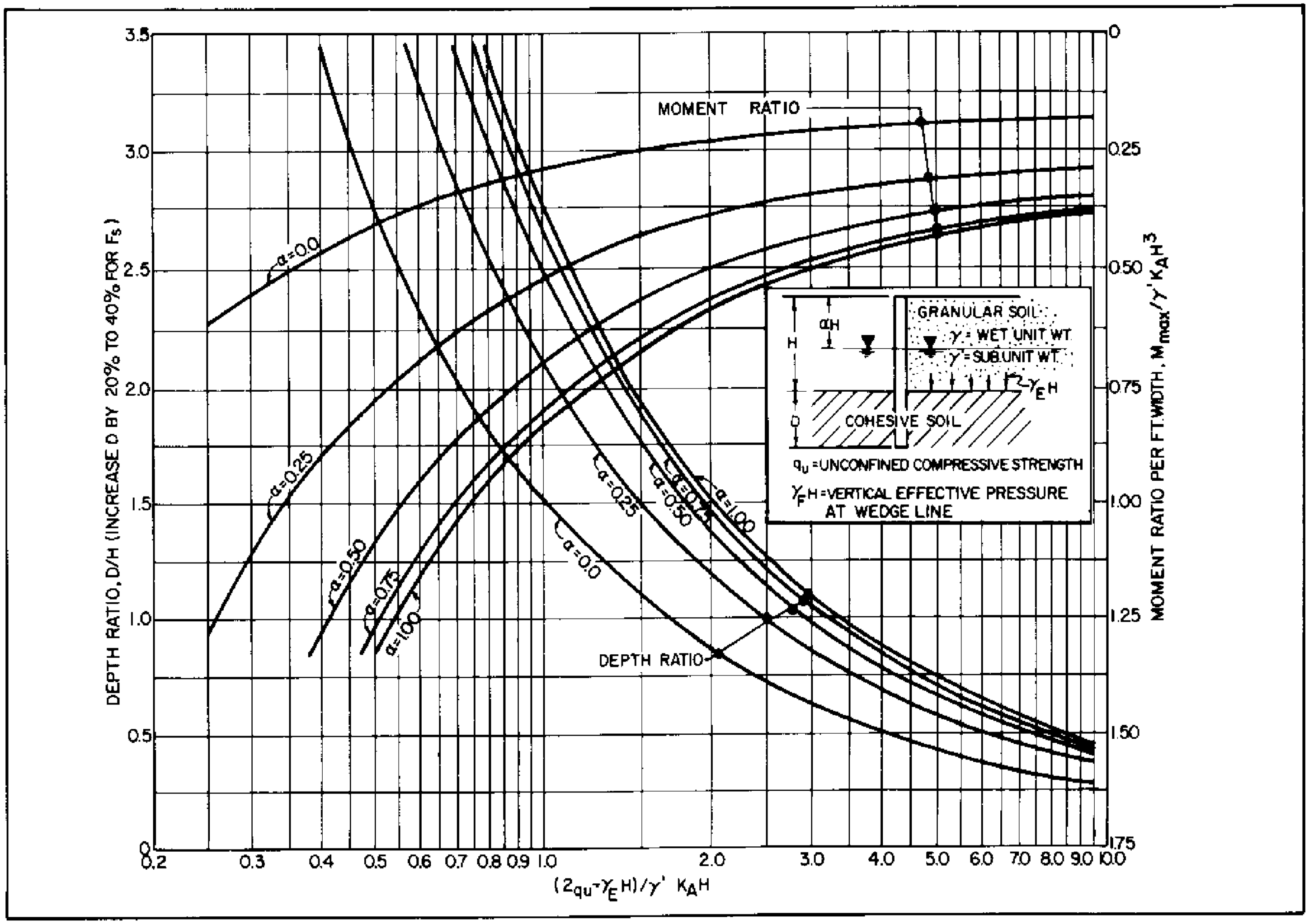


FIGURE 25
Cantilever Steel Sheet Pile Wall in Cohesive Soil with Granular Backfill

EXAMPLE

Backfill: $\phi = 30°$, $\gamma = 120$ pcf, $\gamma' = 60$ pcf

Underlying Cohesive Stratum: $C = 750$ psf, $\gamma' \approx 60$ psf

Depth H to mud line = 20 ft

Depth to water = 5 ft

$\alpha = 5/20 = 0.25$

Wall friction = 0.3 (Table 1)

$K_A = 0.31$ (Figure 5)

$\gamma_E H = 120 \times 5 + 15 \times 60 = 1{,}500$ psf

$q_u = 2C = 1{,}500$ psf

USING FIGURE 25:

$$\frac{2q_u - \gamma_E H}{\gamma' K_A H} = \frac{3000 - 1500}{60 \times 0.31 \times 20} = 4.03$$

Depth ratio, $\frac{D}{H} = 0.69$

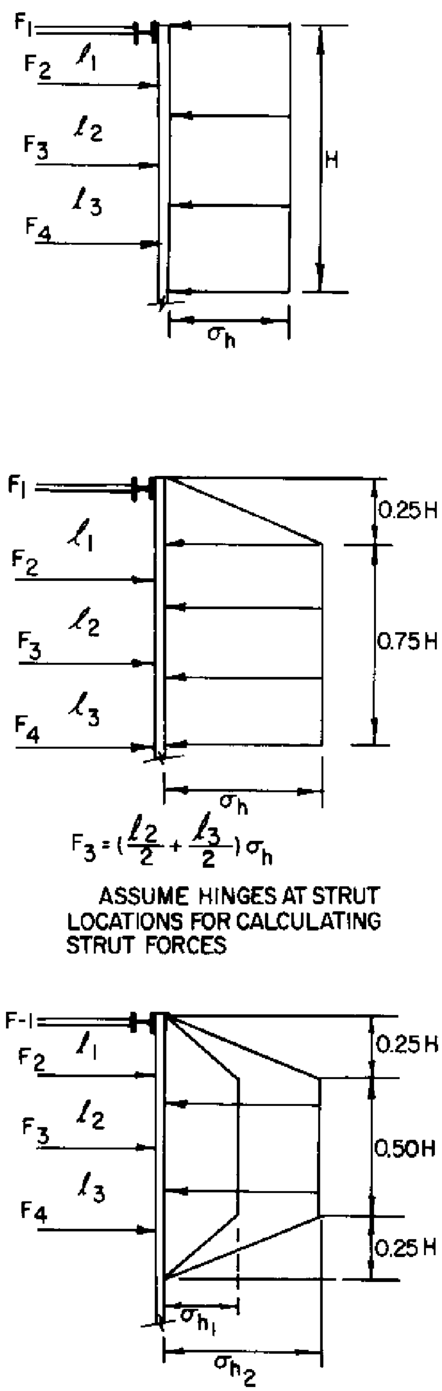
D calculated = $0.69 \times 20 = 13.8$ ft

D design = $13.8 \times 1.3 = 17.9$ ft

Moment ratio = 0.33

$M_{max} = 0.33 \times 60 \times 0.31 \times (20)^3 = 49{,}104$ ft-lb/ft of wall

FIGURE 25 (continued)
Cantilever Steel Sheet Pile Wall in Cohesive Soil with Granular Backfill



(a) SAND

$\sigma_h = 0.65\ K_A \cdot \gamma H$

WHERE $K_A = \tan^2(45 - \phi/2)$

(b) SOFT TO MEDIUM CLAY
($N_o > 6$)

For clays base the selection on
$N_o = \gamma H/c$

$\sigma_h = K_A \cdot \gamma \cdot H$

$K_A = 1 - m \frac{4c}{\gamma H}$;
m = 1 except where cut is underlain by deep soft normally consolidated clay, then $m = 0.4F_{SB}$

See Figure 28 for Factor of Safety against bottom instability, (F_{SB}): $1 \leq F_{SB} \leq 1.5$

(c) STIFF CLAY
($N_o < 4$)
For $4 < N_o < 6$, use larger of diagrams (b) and (c).
$\sigma_{h_1} = 0.2\,\gamma H$; $\sigma_{h_2} = 0.4\,\gamma H$
Use lower value when movements are minimal and short construction period.

FIGURE 26
Pressure Distribution for Brace Loads in Internally Braced Flexible Walls

a. Wall with Raking Braces. When substantial excavation is made before placing an upper brace, movement of the wall is greatest at the top and pressures approach active values. See Figure 27 for design criteria.

b. Braced Narrow Cuts. When a narrow cut is braced stiffly as excavation proceeds, sheeting is restrained at the top and the wall deflects inward at the base. Design the wall employing the following steps:

(1) Compute factor of safety against bottom instability (Figure 28).

(2) Compute strut forces utilizing the method in upper panel of Figure 27.

(3) Compute required section for wall and wale using method in upper panel of Figure 27. In computing the required wall sections, arching could be accounted for by reducing these pressures somewhat in all but the upper span. A reduction of 80% of the values shown would be appropriate.

(4) Re-compute strut forces and the required sections of wales and wall using the pressure diagram of lower panel of Figure 27 for each construction stage.

(5) Compare strut forces, and required sections computed in Step (4) to those computed in Step (3) and select the larger force or section for design. See example in Figure 31.

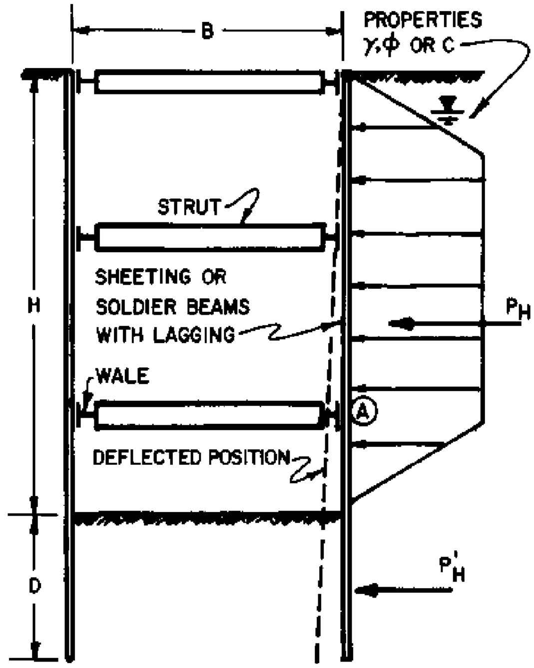
4. TIED BACK FLEXIBLE WALL. Depending on the width of excavation and other factors (see Chapter 1) it may be economical to restrain excavation walls by tie backs. The use of tie backs depends on the existance of subsoils adequate to provide required anchorage. For multi-level tie back systems, drilled in tie backs (i.e. anchors) are usually used. For a single level tie back (e.g., bulkheads), a deadman anchorage, batter pile anchorage or a parallel wall anchorage are usually considered. For details on the drilled anchors - process and hardware, see Reference 6. For details on other anchorage systems see Reference 12 and Reference 13, Foundation Construction, by Carson.

a. Pressure Distribution. For soft to medium clay use a triangular distribution, increasing linearly with depth. For all other soils use a uniform pressure distribution. See Figure 29.

b. Design Procedures. Apply a design procedure similar to internally braced excavation as shown in Figure 27.

5. EXAMPLE OF COMPUTATION. See Figure 30 for example of analysis of braced wall of narrow cut, and Figure 31 for an example of excavation in stages.

6. STABILIZING BERMS. On occasion it is practical to increase the resistance of flexible walls by using stabilizing berms. The lateral resistance of a stabilizing berm will be less than that for an earth mass bounded by a horizontal plane at the top elevation of the berm.



FLEXIBLE WALL OF NARROW CUT

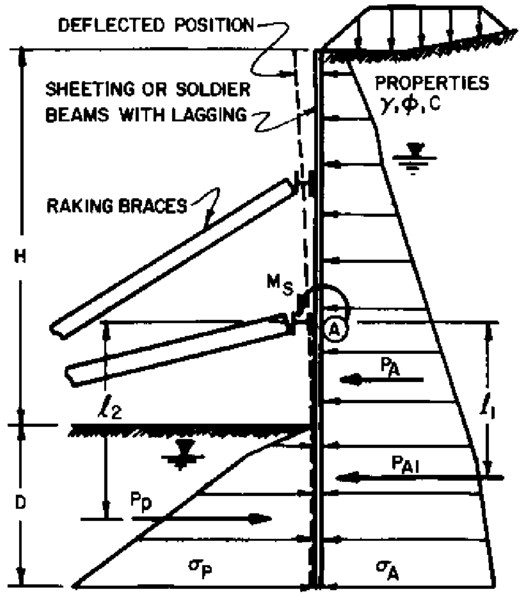
1. COMPUTE PRESSURES ON WALL ABOVE BASE OF CUT BY METHODS OF FIGURE 26. FOR WATER AT BACKFILL SURFACES USE $\gamma = \gamma_{SUB}$ AND ADD PRESSURES FOR UNBALANCED WATER LEVEL. FOR WATER AT BASE OF CUT USE $\gamma = \gamma_T$. INTERPOLATE BETWEEN THESE PRESSURE DIAGRAMS FOR AN INTERMEDIATE WATER LEVEL.

2. DETERMINE STABILITY OF BASE OF CUT BY METHODS OF FIGURE 28. IF BASE IS STABLE, SHEETING TOES IN SEVERAL FEET AND NO FORCE ACTS ON BURIED LENGTH. IF BASE IS UNSTABLE, SHEETING PENETRATES AS SHOWN IN FIGURE 28 AND UNBALANCED FORCE P'_H ACTS ON BURIED LENGTH. IN ANY CASE, PENETRATION MAY BE CONTROLLED BY REQUIREMENT FOR CUT-OFF OF UNDERSEEPAGE.

3. MOMENTS IN SHEETING BETWEEN BRACES = 0.8 x (SIMPLE SPAN MOMENTS), EXCEPT FOR UPPER SPAN WHERE MOMENT = 1.0 x (SIMPLE SPAN MOMENT). MOMENTS IN SHEETING AT POINT A IS COMPUTED FOR CANTILEVER SPAN BELOW A, INCLUDING UNBALANCED FORCE P'_H.

4. REACTION AT BRACES COMPUTED ASSUMING SIMPLE SPAN BETWEEN BRACES.

FIGURE 27
Design Criteria for Braced Flexible Walls



P_A = RESULTANT ACTIVE PRESSURE

P_{A1} = RESULTANT ACTIVE BELOW POINT (A)

FLEXIBLE WALL WITH RAKING BRACES

1. COMPUTE ACTIVE AND PASSIVE PRESSURES BY METHODS IN SECTION 2. PASSIVE PRESSURES FOR CLEAN, COARSE-GRAINED SOILS INCLUDE WALL FRICTION (δ), TABLE 1. IGNORE WALL FRICTION FOR PASSIVE PRESSURES IN OTHER SOIL TYPES AND FOR ACTIVE PRESSURES IN ALL SOILS.

2. MAXIMUM MOMENTS IN SHEETING AND MAXIMUM LOADS IN BRACES ARE USUALLY OBTAINED AT A CONSTRUCTION STAGE WHEN EXCAVATION FOR A BRACE AND WALE IS COMPLETE AND JUST PRIOR TO PLACING THE BRACE. FOR EACH SUCCESSIVE STAGE OF EXCAVATION COMPUTE SHEETING MOMENTS AND BRACE LOADS BY ASSUMING SIMPLE SPAN BETWEEN LOWEST BRACE THEN IN PLACE AND POINT OF ZERO NET PRESSURE BELOW EXCAVATION.

3. FOR TEMPORARY CONSTRUCTION CONDITIONS, APPLY FACTOR OF SAFETY OF 1.5 TO COMPUTE PASSIVE PRESSURES. TO ALLOW FOR POSSIBLE CONSTRUCTION SURCHARGE AND RIGIDITY OF UPPER BRACE POINT, INCREASE LOAD ON UPPER WALE AND BRACE BY 15% OF COMPUTED VALUE.

4. REQUIRED PENETRATION OF SHEETING BELOW FINAL SUBGRADE GENERALLY IS CONTROLLED BY CONDITIONS AT COMPLETION OF EXCAVATION. PENETRATION REQUIRED IS DETERMINED BY EQUILIBRIUM OF FREE ENDED SPAN BELOW POINT (A). ASSUMING FIXITY AT POINT (A):

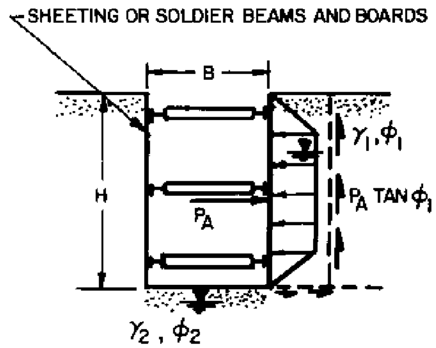
$$P_{A1}\ell_1 - \frac{P_p}{F_s}\ell_2 - M_S = 0$$

M_S = ALLOWABLE MOMENT IN SHEETING

5. CHECK POSITIVE MOMENTS IN SPAN BELOW POINT (A) FOR THIS FINAL LOADING CONDITION.

FIGURE 27 (continued)
Design Criteria for Braced Flexible Walls

CUT IN COHESIONLESS SOIL



STABILITY IS INDEPENDENT OF H AND B, BUT VARIES WITH γ, ϕ AND SEEPAGE CONDITION.

SAFETY FACTOR, $F_S = 2N_{\gamma 2}\left(\frac{\gamma_2}{\gamma_1}\right)K_A \tan\phi$

$N_{\gamma 2}$ = BEARING CAPACITY FACTOR, FIGURE 1, CHAPTER 4

IF GROUNDWATER IS AT A DEPTH OF (B) OR MORE BELOW BASE OF CUT:

γ_1 AND γ_2 ARE TAKEN AS MOIST UNIT WEIGHT

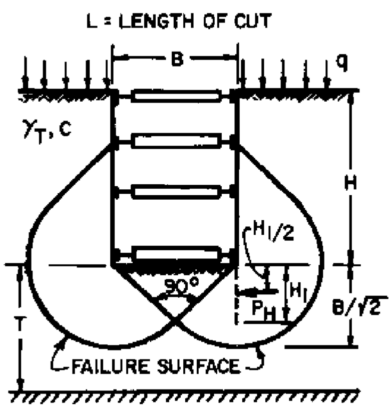
IF GROUNDWATER IS STATIC AT BASE OF CUT:

γ_1 = MOIST WEIGHT, γ_2 = SUBMERGED WEIGHT.

IF SEEPAGE IS MOVING UPWARD TO BASE OF CUT:

γ_2 = (SATURATED UNIT WEIGHT) − (UPLIFT PRESSURE)

CUT IN CLAY, DEPTH OF CLAY UNLIMITED (T > 0.7B)



IF SHEETING TERMINATES AT BASE OF CUT:

SAFETY FACTOR, $F_S = \dfrac{N_C\,C}{\gamma_T H + q}$

N_C = BEARING CAPACITY FACTOR, FIGURE 2, CHAPTER 5 WHICH DEPENDS ON DIMENSIONS OF THE EXCAVATION: B, L AND H (USE H = Z).

C = UNDRAINED SHEAR STRENGTH OF CLAY IN FAILURE ZONE BENEATH AND SURROUNDING BASE OF CUT.

q = SURFACE SURCHARGE.

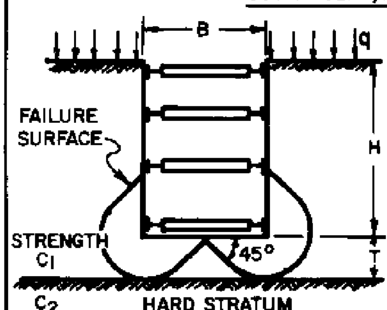
IF SAFETY FACTOR IS LESS THAN 1.5, SHEETING MUST BE CARRIED BELOW BASE OF CUT TO INSURE STABILITY.

FORCE ON BURIED LENGTH:

IF $H_1 > \frac{2}{3}\frac{B}{\sqrt{2}}$, $P_H = .7\,(\gamma_T HB - 1.4\,CH - \pi CB)$

IF $H_1 < \frac{2}{3}\frac{B}{\sqrt{2}}$, $P_H = 1.5H_1\left(\gamma_T H - \frac{1.4CH}{B} - \pi C\right)$

CUT IN CLAY, DEPTH OF CLAY LIMITED BY HARD STRATUM (T ≤ 0.7B)



SHEETING TERMINATES AT BASE OF CUT. SAFETY FACTOR:

CONTINUOUS EXCAVATION; $F_S = N_{CD}\,\dfrac{C_1}{\gamma_T H + q}$

RECTANGULAR EXCAVATION; $F_S = N_{CR}\,\dfrac{C_1}{\gamma_T H + q}$

N_{CD} AND N_{CR} = BEARING CAPACITY FACTORS.

FIGURE 5 CHAPTER 4, WHICH DEPEND ON DIMENSIONS OF THE EXCAVATION: B, L AND H, (USE H = Z)

NOTE: IN EACH CASE FRICTION AND ADHESION ON BACK OF SHEETING IS DISREGARDED. CLAY IS ASSUMED TO HAVE A UNIFORM SHEAR STRENGTH = C THROUGHOUT FAILURE ZONE.

FIGURE 28
Stability of Base for Braced Cut

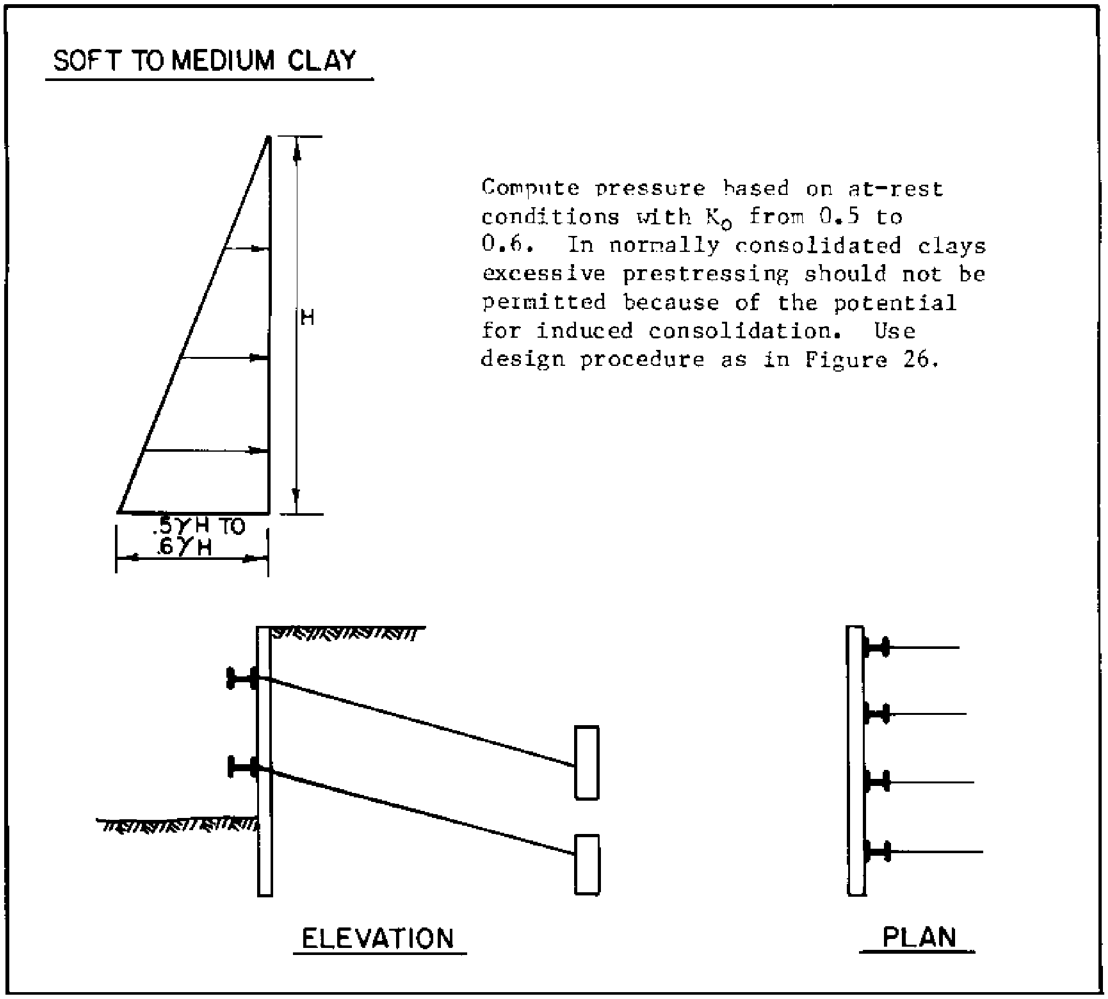
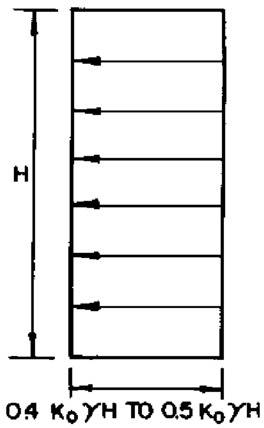


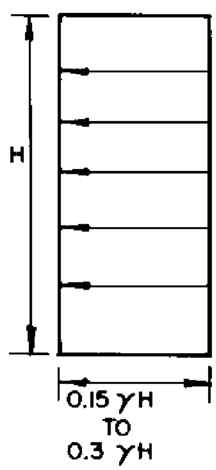
FIGURE 29
Pressure Distribution for Tied-Back Walls

SANDS



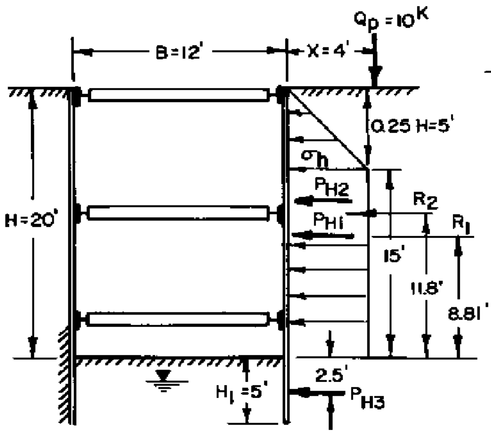
Where deformations are critical and tie-backs are prestressed to 100% of desig load, compute pressure based on at-rest conditions. Use K_o = 0.4 for dense sand, and K_o = 0.5 for loose sand.

STIFF TO VERY STIFF CLAY



Use pressure ordinate to produce the same force as for braced excavation. 0.3 is applicable for stability number of about 4, and 0.15 is applicable when stability number is less than 4. Use design procedure as in Figure 26.

FIGURE 29 (continued)
Pressure Distribution for Tied-Back Walls



GIVEN CONDITIONS:

EXCAVATION IN SILTY CLAY.
$C = 400$ PSF, $\phi = 0$, $\gamma_T = 120$ PCF
LENGTH OF EXCAVATION, L = 80'

DETERMINE: PRESSURES ON WALL, FORCE ON BURIED LENGTH OF SHEETING AND STABILITY OF BASE OF CUT.

STABILITY OF BASE OF CUT (SEE FIGURE 28)

$F_{SB} = \dfrac{N_C C}{\gamma_T H + q}$, $q = 0$ (NO UNIFORM SURCHARGE)

FOR N_C, (FIGURE 2, CHAPTER 5) $\dfrac{H}{B} = \dfrac{Z}{B} = \dfrac{20}{12} = 1.67$,

$\dfrac{B}{L} = \dfrac{12}{80} = 0.15$, $N_{CC} = 6.9$

$N_{CR} = N_{CC}(1 + 0.2\ B/L) = 6.9(1 + 0.2(0.15)) = 7.1$

$F_S = \dfrac{7.1 \times 400}{120 \times 20 + 0} = 1.18 < 1.5$

DRIVE SHEETING BELOW BOTTOM OF EXCAVATION

PRESSURE ON WALL FROM SURROUNDING SOIL (SEE FIGURE 26)

$K_A = 1 - m\ \dfrac{4C}{\gamma H}$: $m = 0.4\ F_{SB} = 0.4 \times 1.18 = 0.47$

$= 1 - (0.47)\left(\dfrac{4 \times 400}{120 \times 20}\right) = 0.69$

$\sigma_h = K_A \gamma H = 0.69 \times 0.12 \times 20 = 1.66$ KSF

$P_{H1} = \dfrac{(15+20)(1.66)}{2} = 29.05$ KIPS

LOCATION OF RESULTANT:

$R_1 = \dfrac{1.66 \times 5/2 \times (15 + 5/3) + 1.66 \times 15 \times 15/2}{29.05} = 8.81'$

PRESSURES ON WALL FROM SURCHARGE (SEE FIGURE 11)

$m = \dfrac{X}{H} = \dfrac{4}{20} = 0.2$

$P_{H2} = .78\ \dfrac{Q_p}{H} = .78\ \dfrac{10}{20} = .39$ KIP

LOCATION OF RESULTANT:

$R_2 = .59H = .59 \times 20 = 11.8'$

FORCE ON BURIED LENGTH OF SHEETING: (SEE FIGURE 28)

ASSUME $H_1 = 5 < \dfrac{2}{3}\ \dfrac{B}{\sqrt{2}}$, FOR T > 0.7B RESULTANT FORCE P_{H3}:

$P_{H3} = 1.5\ H_1 \left(\gamma_T H - \dfrac{1.4CH}{B} - \pi C\right)$

$P_{H3} = 1.5 \times 5 \left(0.12 \times 20 - \dfrac{1.4 \times .4 \times 20}{12} - 3.14 \times 4\right) = 1.6$ KIP

NOTE: ALL COMPUTATIONS ARE PER LINEAR FOOT OF WALL.

FIGURE 30
Example of Analysis of Pressures on Flexible Wall of Narrow Cut In Clay - Undrained Conditions

ASSUMPTIONS

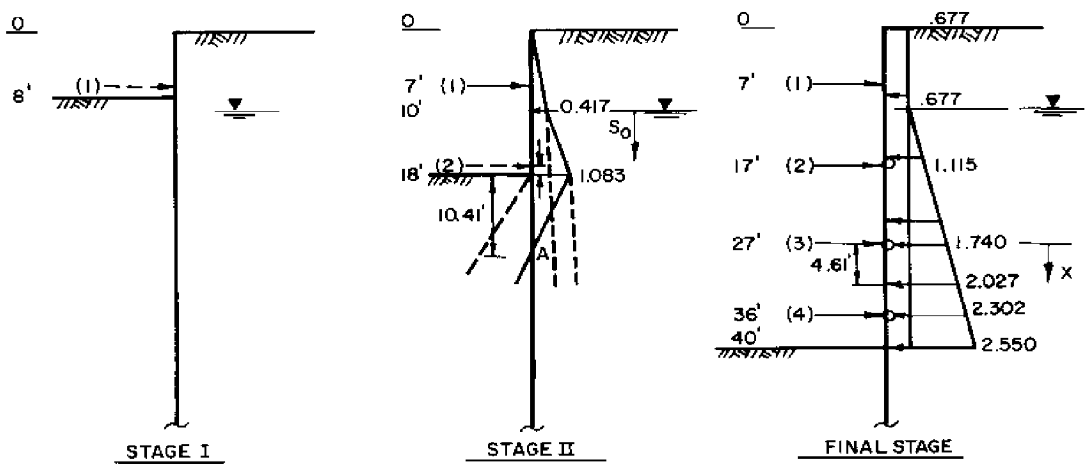
1. NO SURCHARGE LOAD. 2. NO WALL FRICTION

PROPERTIES

ϕ = 30° γ = 0.125 KCF γ' = 0.0625 KCF

c = 0 DEPTH OF EXCAVATION 40'

GWL = 10' BELOW GROUND LEVEL



COMPUTATIONS

FROM FIGURE 2

$K_A = 1/3$, $K_P = 3$

A. STAGE I

(PRIOR TO INSTALLATION OF BRACE 1)

SHEETING ACTS AS CANTILEVER WALL.

USE FIGURE 24

$\alpha = 0$ $K_P/K_A = 9$

D/H = 0.95 $\therefore$ REQUIRED D = 0.95 x 8 x 1.4 = 10.64' < 40'

$M_{MAX}/\gamma' K_A \cdot H^3 = 0.37$ M_{MAX} = 3.946 FT-KIP

FIGURE 31
Example of Excavation in Stages

B. STAGE II

1. ACTIVE PRESSURE

AT WATER LEVEL, σ_A (10) = 1/3 x 0.125 x 10 = 0.417 KSF

AT EXCAVATION LEVEL, σ_A (18) = 0.417 + 1/3 x 0.0625 x 8 = 0.583 KSF

WATER PRESSURE ON ACTIVE SIDE

P_W (18) = 0.0625 x 8 = 0.500 KSF

TOTAL PRESSURE (18) = σ_A (18) + P_W (18) = 1.083 KSF

2. POINT OF ZERO NET PRESSURE

APPLY F_S = 1.5 TO K_P

SLOPE OF THE NET PRESSURE DIAGRAM = (3/1.5 - 1/3) 0.0625 = 0.104

$$\text{DISTANCE TO (A)} = \frac{1.083}{0.104} = 10.41 \text{ FT}$$

3. REACTION AT (1) AND (A) PER LINEAR FOOT OF WALL

ASSUME HINGE (ZERO BENDING MOMENT) AT (A)

$$R(1) = \left[(1.083 \times 10.41 \times 1/2 \times (2/3 \times 10.41) + (1.083 - 0.417) \times 8 \times 1/2 \times (10.41 + 8/3) + 0.417 \times 8 \times (10.41 + 8/2) + 0.417 \times 10 \times 1/2 \times (10.41 + 8 + 10/3)\right] \times \frac{1}{(10.41 + 11)}$$

R(1) = 7.817 K, USE R(1) = 1.15 x 7.817 = 8.99 K $\simeq$ 9.0 K

R (A) = 5.905 K $\simeq$ 5.9 K

4. POINT OF ZERO SHEAR

TRY A LOCATION BETWEEN BOTTOM OF EXCAVATION (DEPTH 18') AND (1)

$$7.817 - (1/2 \times 10 \times 0.417) - (S_0 \times 0.417) - (1/2 \times S_0 \times \frac{0.666}{8} S_0) = 0, \quad S_0 = 7.75'$$

5. MAXIMUM MOMENT

$$M_{MAX.} = \left[7.817 \times (7.75 + 3)\right] - \left[(1/2 \times 10 \times 0.417) \times (7.75 + \frac{10}{3})\right] - \left[(7.75 \times 0.417) \times \frac{7.75}{2}\right] - \left[(1/2 \times 7.75^2 \times \frac{0.666}{8}) \times \frac{7.75}{3}\right] = 41.9 \text{ FT-KP}$$

C. FINAL STAGE

1. PRESSURE DISTRIBUTION

USE PRESSURE DIAGRAM FROM FIGURE 26

γ_{av} = 0.25 x 0.125 + 0.75 x 0.0625 = 0.0781 = 1 KSF

σ_h = 0.65 x 1/3 x 0.0781 x 40 = 0.677 KSF

P_W (30) = 0.0625 x 30 = 1.875 KSF

FIGURE 31 (continued)
Example of Excavation in Stages

2. STRUT LOADS PER LINEAR FOOT OF WALL

$R(1) = \left[0.677 \times 17^2/2 + 0.0625 \times 7^2/2 \times 7/3\right] 1/10 = 10.14\ K$, FOR DESIGN $R1 = 1.15 \times 10.14 = 11.66\ K$

$R(2) = \Big(\big[(0.677 \times 17) + (1/2 \times (1.115 - 0.677) \times 7) - 10.14)\big] + \big((1.115 \times 10 \times 10/2) +$
$(1/2 \times (1.740 - 1.115) \times 10 \times 10/3)\Big)\ 1/10 = 9.52\ K$

$R(3) = \Big(\big[(0.677 \times 27) + (1/2 \times (1.740 - 0.677) \times 17) - 10.14 - 9.52\big] + \big((1.740 \times 9 \times 9/2) +$
$(1/2 \times (2.302 - 1.740) \times 9 \times 9/3)\Big)\ 1/9 = 16.33\ K$

$R(4) = \Big(\big[(36 \times 0.677) + (1/2 \times (2.302 - 0.677) \times 26) - 10.14 - 9.52 - 16.33\big] + \big(2.302 \times 4 \times 4/2) +$
$(1/2 \times (2.550 - 2.302) \times 4 \times 4/3)\Big)\ 1/4 = 14.27\ K$

3. MOMENT

MAXIMUM MOMENT IS LIKELY TO OCCUR BETWEEN (3) AND (4).

POINT OF ZERO SHEAR FROM (3).

$$\overbrace{\left[(1.74 \times 9 \times 9/2) + 1/2 \times (2.302 - 1.74) \times 9 \times 9/3\right] \times 1/9}^{8.67} = 1.74\,(x) + (1/2)\left(\frac{2.302 - 1.74}{9}\right)(x)(x)$$

$x - 4.61$ FT., $\sigma = 2.027$ KSF

$M_{MAX} = (8.67 \times 4.61) - (1.74 \times 4.61 \times \frac{4.61}{2}) - 1/2 \times (2.027 - 1.74) \times 4.61 \times \frac{4.61}{3}$
$= 20.5$ FT KIP; $M_{DESIGN} = 0.8 \times 20.5 = 16.4$ FT-KIP.

D. SUMMARY

CONSTRUCTION STAGE	STRUT LOADS KIP	MOMENTS FT - KIP
I	—	3.95
II	R(1) = 8.99	41.9 BETWEEN (1) AND (4)
FINAL	R(1) = 11.66; R(2) = 9.52 R(3) = 16.33; R(4) = 14.27	16.4 BETWEEN (3) AND (4)

NOTE: (A) THE MOMENT AT STAGE II IS GREATER THAN THE FINAL MOMENT. INTERMEDIATE STAGES MUST ALSO BE CHECKED AS PER PROCEDURE IN FIGURE 27.

(B) IF SIMPLE AREA METHOD IS SELECTED FOR THE COMPUTATIONS OF LOADS IN STRUTS (1) AND (2), THEN LOAD IN (1) WILL DECREASE AND (2) WILL INCREASE.

FIGURE 31 (continued)
Example of Excavation in Stages

E. PENETRATION BELOW SUBGRADE

1. PRESSURE COMPUTATION

ACTIVE σ_A (10') = 1/3 x 0.125 x 10 = 0.417 KSF

σ_A (36') = 1/3 x 0.125 x 10 + 1/3 x 0.0625 x 26 = 0.958 KSF

σ_A (40') = 1/3 x 0.125 x 10 + 1/3 x 0.0625 x 30 = 1.042 KSF

WATER PRESSURE (UNBALANCED HEAD)

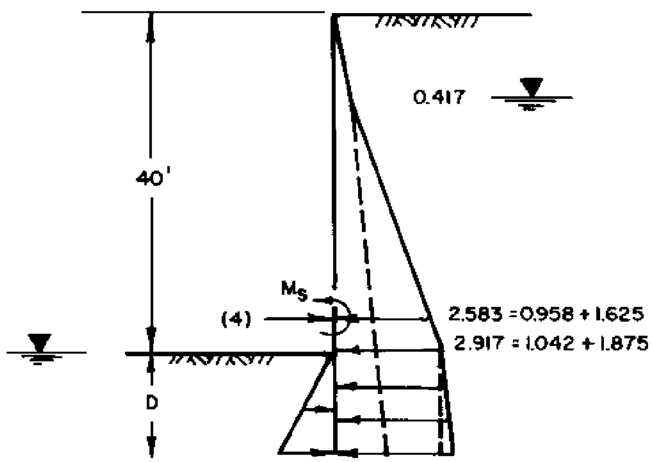
P_W (36') = 0.0625 x 26 = 1.625 KSF

P_W (40') = 0.0625 x 30 = 1.875 KSF

PASSIVE PRESSURE

σ_P (D) = $\frac{3}{1.5}$ x 0.0625 x D = 0.125 D

(APPLY F_S = 1.5 TO PASSIVE PRESSURE)



2. DEPTH REQUIREMENT TO LIMIT MOMENT IN SHEETING (SEE FIGURE 27 (CONTINUED), 4.)

USE PZ 27 S = 30.2 IN^3/FT OF WALL

USE σ_a = 27,000 PSI

ALLOWABLE MOMENT = $\frac{30.2 \times 27}{12}$ = 67.95 FT-KIP = M_S

TAKE MOMENTS ABOUT (4) TO DETERMINE D.

$67.95 + \left(1/2 \times 0.125\,D \times D \times (4 + 2/3\,D)\right) - 2.583 \times 4 \times 4/2 - 1/2 \times (2.917 - 2.583) \times 4 \times 2/3 \times 4$

$- 2.917 \times D \times (4 + D/2) - 1/2 \times (1/3 \times 0.0625)\,D \times D \times (4 + 2/3\,D) = 0$

$D^3 - 35.3\,D^2 - 332.5\,D + 1296.6 = 0$

$D \approx 3$ FT.

3. DEPTH REQUIREMENT FOR CONTROL OF PIPING. (DM-7.1, CHAPTER 6)

ASSUME W/H_W = 1.5 (I.E., WIDE EXCAVATION)

F_S = 1.2

D/H_W = 0.65

OR D = 30 x 0.65 = 19.5 FT

HENCE PIPING GOVERNS THE DEPTH OF PENETRATION FOR THE SHEETING.

FIGURE 31 (continued)
Example of Excavation in Stages

a. Method of Analysis. Perform wedge force equilibrium for several trial failure surfaces, and plot corresponding values of horizontal resistance for each trial failure surface. The minimum value of horizontal resistance obtained from the curve is the total passive earth pressure for the berm. An approximate method of analysis is to replace the berm with an equivalent sloping plane, and assign an appropriate passive pressure coefficient.

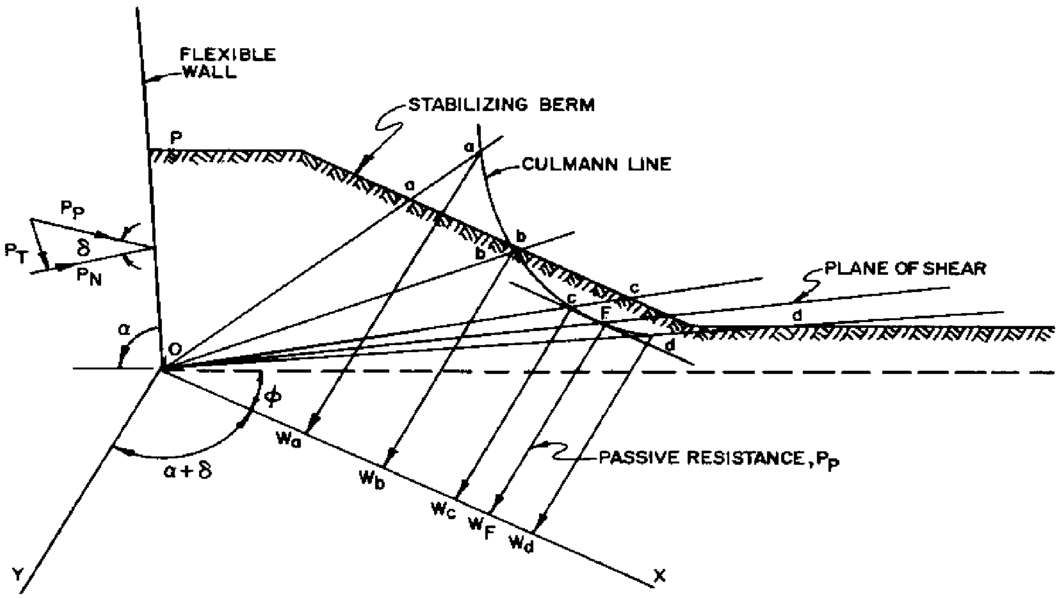
b. Graphic Procedure. A graphic procedure (Culmann Method) for evaluating the lateral resistance for granular soils is given in Figure 32.

7. SOLDIER PILES. A frequently used internal bracing system consists of soldier piles with lagging. The passive earth resistance acting on individual soldier piles may be computed as shown in Figure 33. For cohesive soils use uniform resistance of 2c neglecting the soil resistance to a depth of 1.5 times the pile width b from the bottom of the excavation. For granular soils, determine K+p, without wall friction and neglect the soil resistance to a depth equal to b below the bottom of the excavation. Total resisting force is computed by assuming the pile to have an effective width of 3b, for all types of soils. This is because the failure in soil due to individual pile elements is different from that of continuous walls for which pressure distributions are derived.

8. GABION STRUCTURES. As illustrated in Figure 34, gabions are compartmented, rectangular containers made of heavily galvanized steel or polyvinylchloride (PVC) coated wire, filled with stone from 4 to 8 inches in size, and are used for control of bank erosion and stabilization. When water quality is in doubt (12<pH<6) or where high concentration of organic acid may be present, PVC coated gabions are necessary. At the construction site, the individual gabion units are laced together and filled with stone.

a. Design. Gabions are designed as mass gravity structures (see Figure 15). When designing a vertical face wall it should be battered at an angle of about 6 deg. to keep the resultant force toward the back of the wall. The coefficient of friction between the base of a gabion wall and a cohesionless soil can be taken as tan[phi] for the soil. The angle of wall friction, [delta] may be taken as 0.9[phi]. Where the retained material is mostly sand, a filter cloth or granular filter is recommended to prevent any leaching of the soil. Determine the unit weight of gabions by assuming the porosity to be 0.3. Specific gravity of common material ranges between 2.2 (sandstone) and 3.0 (basalt). Along all exposed gabion faces the outer layer of stones should be hand placed to ensure proper alignment, and a neat compact square appearance.

b. Cohesive Soils. A system of gabion counterforts is recommended when designing gab on structures to retain clay slopes. They should be used as headers and should extend from the front of the wall to a point at least one gabion length beyond the critical slip circle of the bank. Counterforts may be spaced from 13 feet (very soft clay) to 30 feet (stiff clay). A filter is also required on the back of the wall so that clay will not clog the free draining gabions.



1. Draw berm to scale.
2. Layout OX from point 0 at angle θ below horizontal.
3. Layout OY from point 0 at angle ($\alpha + \delta$) below OX.
4. Assume failure surfaces originating at point 0 and passing through points a, b, c, etc.
5. Compute the weight of each failure wedge.
6. Layout the weight of each failure wedge along OX to a convenient scale.
7. Draw a line parallel to OY for each failure wedge from its weight plotted on OX to its failure plane (extrapolated where necessary).
8. Connect the intersecting points from 7 above with a smooth curve - this is the Culmann Curve. Draw a tangent to this curve which is also parallel to OX.
9. Through the tangent point F, draw a line parallel to OY to intersect OX at W_F. Distance FW_F is the value of P_p in the weight scale.
10. Normal component of the passive resistance, $P_N = P_p \cos \delta$.
11. To compute pressure distribution on the wall, assume a triangular distribution.

Figure 32
Culmann Method for Determining Passive Resistance of Earth Berm
(Granular Soil)

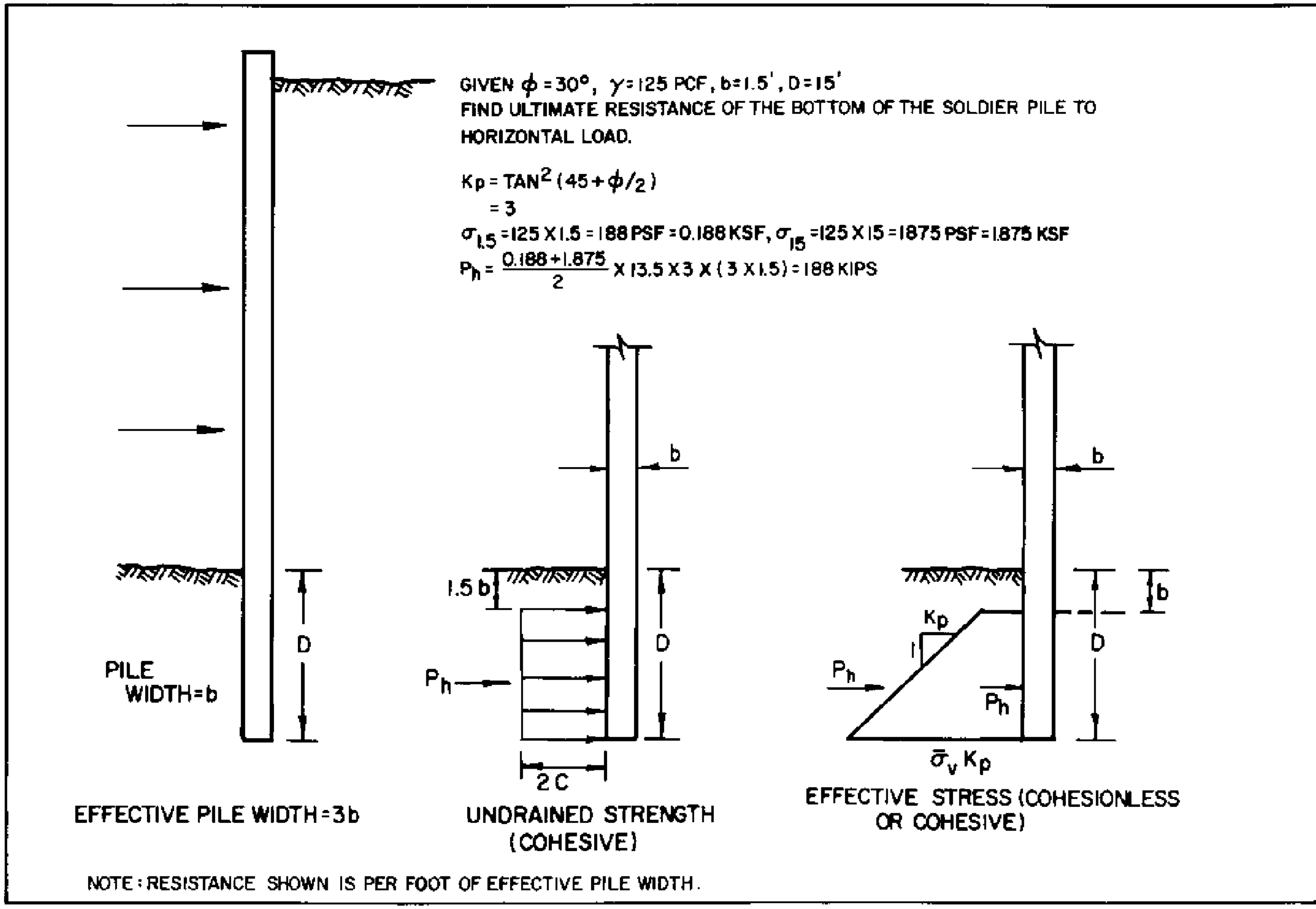
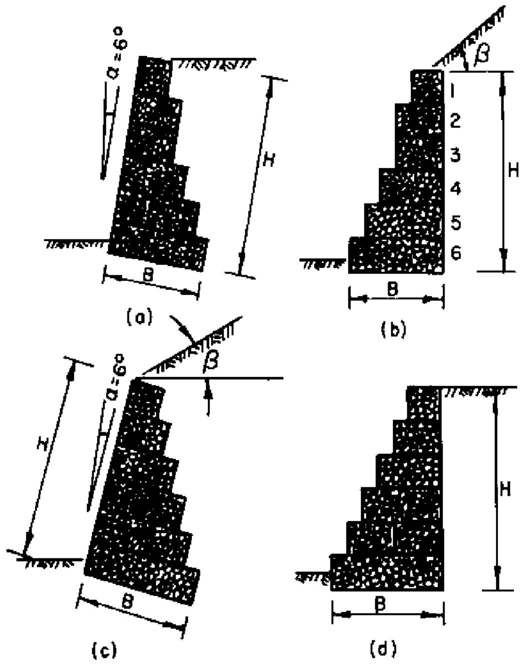


FIGURE 33
Passive Pressure Distribution for Soldier Piles



Gabion Retaining Wall

Types - Common Gabion walls shown on accompanying diagrams are:

a) Battered face wall with horizontal backfill.
b) Stepped face wall with sloped backfill.
c) Battered face wall with sloped backfill.
d) Stepped face wall with horizontal backfill.

The choice of either battered or stepped faces rests with designer; stepped face recommended if wall is more than 10 feet high.

Gabion Fill - Hard, durable, clean stone 4 to 8 inches in size or other approved size.

Design: Design criteria for gravity walls apply. Wall section resisting overturning and sliding. To increase wall stability, recommended to tilt the wall at an angle of 6° (i.e. 1:10).

The angle of friction between the base of gabion wall and granular soil may be assumed 0.9 times the angle of internal friction of soil.

For retaining clay slopes, a system of gabion counterforts is recommended.

Compute active soil pressure behind the wall using Coulomb Wedge theory and design mass of the wall to balance the force exerted by that soil wedge. (Higher than active pressures may be used depending on compaction conditions and limitations on deformations.)

Maximum pressure at the base of gabion wall must be less than the anticipated bearing capacity of the soil under the wall.

When water quality is in doubt (pH below 6 or greater than 12) or where high concentration of organic acids may be present, use of PVC (polyvinylchloride) coated gabions is recommended.

FIGURE 34
Gabion Wall

9. REINFORCED EARTH. Reinforced earth is a system of tying vertical facing units into a soil mass with their tensile strips. It consists of four elements: (1) a soil backfill, (2) tensile reinforcing strips, (3) facing elements at boundaries, and (4) mechanical connections between reinforcements and facing elements. The soil backfill is generally granular material with not more than 15% by weight passing a No. 200 mesh sieve. It should not contain materials corrosive to reinforcing strips. Reinforcing strips include smooth and rough strips of non-corrodable metals or treated metals about 3 inches wide. Facing consists of steel skin or precast concrete panels about 7 inches thick.

A wall constructed of reinforced earth is a gravity wall and its safety should be checked as in Figure 15.

Internal safety of reinforced earth is checked as illustrated in Figure 35. For further guidance on reinforced earth see Reference 14, <u>Reinforced Earth Retaining Walls</u>, by Lee, et al. and Reference 15, <u>Symposium of Earth Reinforcement, Proceedings of a Symposium</u>, by American Society of Civil Engineers.

10. EARTH FILLED CRIB WALLS. See Figure 36 (Reference 16, <u>Concrete Crib Retaining Walls</u>, by Portland Cement Association) for types and design criteria. For stability against external forces, a crib wall is equivalent to gravity retaining wall (Figure 15). For design of structural elements, see Reference 17, <u>Foundations, Design and Practice</u>, by Seelye.

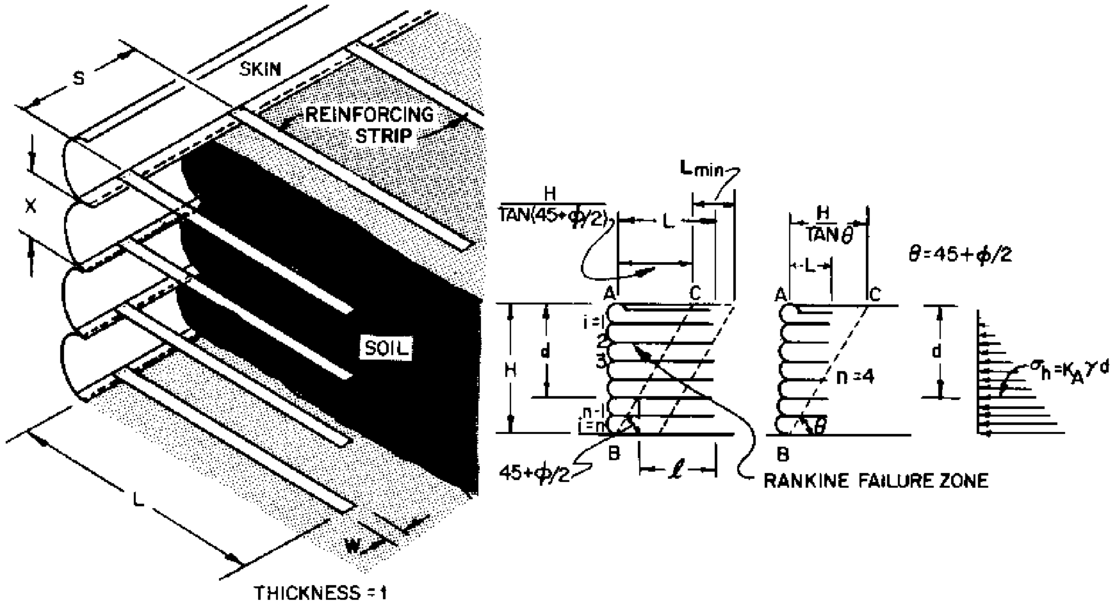
Section 5. COFFERDAMS

1. TYPES. Double-wall or cellular cofferdams consist of a line of circular cells connected by smaller arcs, parallel semi-circular walls connected by straight diaphragms, or a succession of cloverleaf cells (see Figure 37). For analysis, these configurations are transformed into equivalent parallel wall cofferdams of width B.

2. ANALYSIS. Stability depends on ratio of width to height, the resistance of an inboard berm, if any, and type and drainage of cell fill materials.

a. <u>Exterior Pressures.</u> Usually active and passive pressures act on exterior faces of the sheeting. However, there are exceptions to this and these are illustrated in Figure 37.

b. <u>Stability Requirements</u>. A cell must be stable against sliding on its base, shear failure between sheeting and cell fill, shear failure on centerline of cell, and it must resist bursting pressures through interlock tension. These factors are influenced by foundation type. See Figure 37 for design criteria for cofferdams with and without berms, on foundation of rock or of coarse-grained or fine-grained soil. See Reference 18, <u>Design, Construction and Performance of Cellular Cofferdams</u>, by Lacroix, et al., for further guidance.



Safety against breaking of reinforced strips.

$$F_S = \frac{f_s\ W\ t}{K_A\ \gamma\ H\ S\ X}$$

S = Horizontal spacing between strips   X = Vertical Spacing between strips

f_s = allowable stress of reinforced strips.

Typically W = 3". A high factor of safety, F_s = 3.2, is used even though allowable metal stress is utilized in computing strip thickness. This is done to account for unknowns such as durability and corrosion.

SAFETY AGAINST PULLOUT $F_s = \frac{2\ L_{min} . W\ TAN\ \delta}{K_A \cdot S \cdot X}$

L_{min} is measured beyond zone of Rankine failure. The upper strips may not have enough length to fulfill this requirement, but as long as the average length of all the strips satisfies this conditon the wall is considered satisfactory.

d = depth beneath top of wall

t = thickness of strip

γ = unit weight of backfill

B = width of wall

K_A = coefficient of each active pressure (higher than active value may be used depending on compaction conditions and limitations on deformations).

δ = angle of friction between reinforcing strip and the backfill material

ℓ = effective length of tie beyond potential sliding surface

FIGURE 35
Reinforced Earth

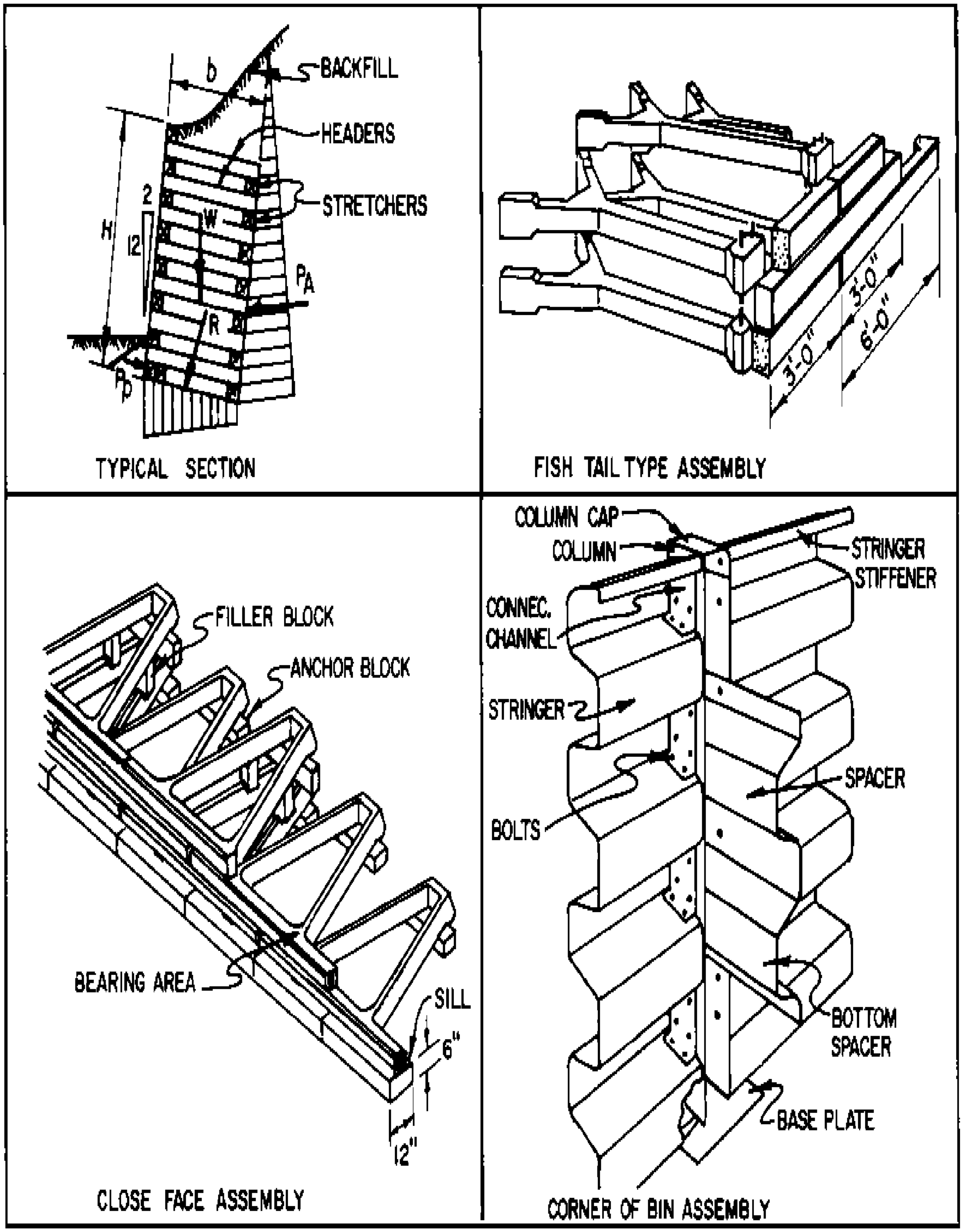


FIGURE 36
Design Criteria for Crib and Bin Walls

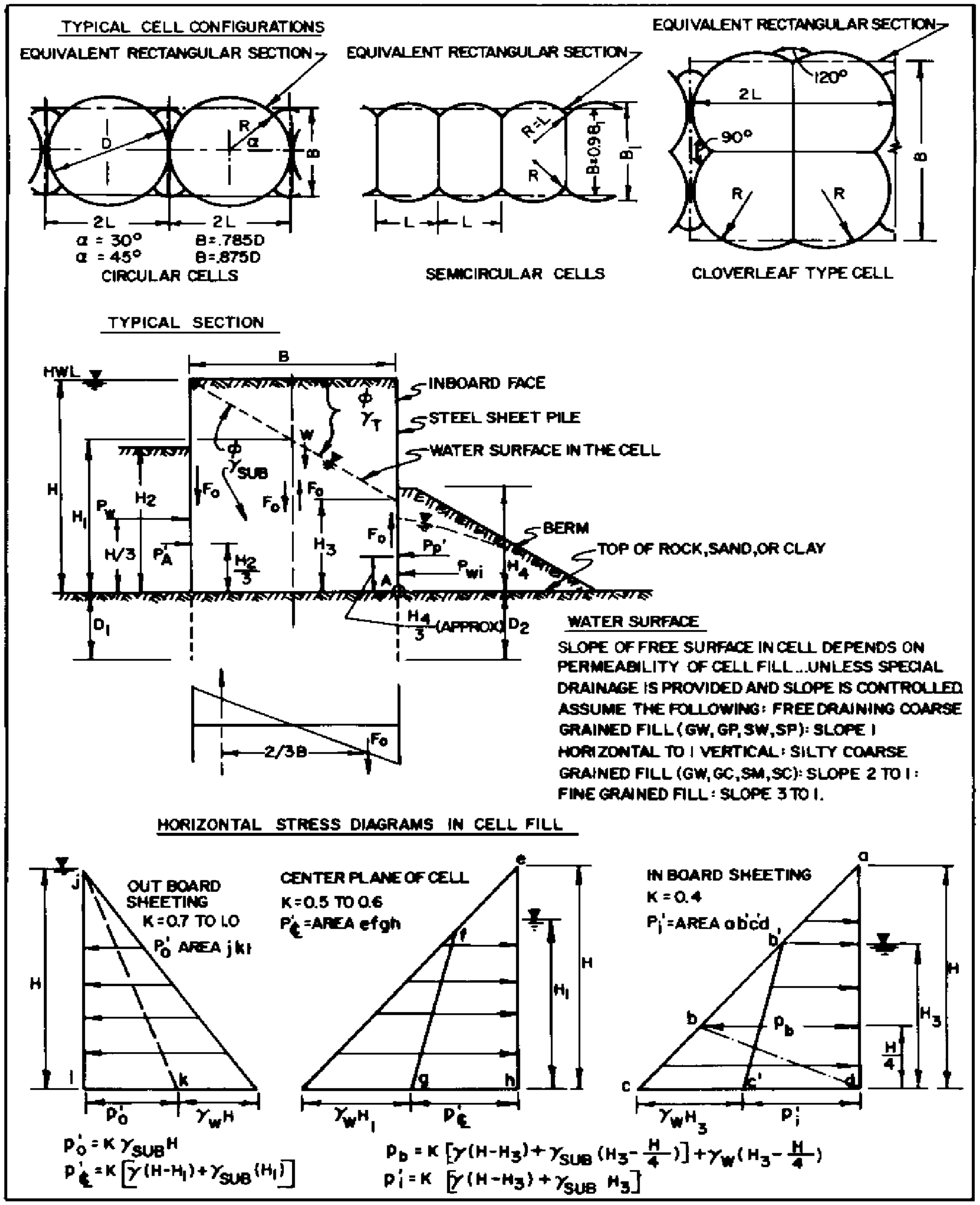


FIGURE 37
Design Criteria for Cellular Cofferdams

PARAMETERS FOR ANALYSIS

1.	Equivalent width of cofferdam.	Assume $B = 0.85H$ for first trial.
2.	Effective weight of cell fill.	$W = [B(H-H_1)\gamma_T + B(H_1)\ \gamma_{sub}]$
3.	Average distance between cross walls.	L
4.	Horizontal active force on outboard side - compute using $K_A = \tan^2(45 - \phi/2)$.	$P_A' = K_A \dfrac{\gamma_{SUB}(H_2)^2}{2}$
5.	Coefficient of horizontal earth pressure.	K (varies - see horizontal pressure - diagram)
6.	Water force on outboard side.	$P_w = \gamma_W \dfrac{(H)^2}{2}$
7.	Horizontal passive force due to berm plus water force.	$P_p = P_p' + P_{wi}$ (include wall friction between sheet pile and soil
8.	Net overturning moment due to total horizontal force.	$M_O = (P_W \times \frac{H}{3}) + (P_A' \times \frac{H2}{3}) - (P_p \times \frac{H4}{3})$ (point of application of P_p is approximated as H4/3, see References in text for further guidance)
9.	Resisting moment due to cell fill.	$M_R = W(B/2)$
10.	Radius of cell wall.	R
11.	Interlock tension.	$T = P_b\ L$ where P_b = total horizontal stress at point b Zone at maximum interlock tension located at H/4 above base. See stress diagram, Inboard Sheeting and references cited in text
12.	Ultimate interlock strength.	T_u = 16 kip/in for ordinary U.S. steel sheet piles and 28 kips/in for high interlock U.S. sheet piles
13.	Effective unit weight.	γ_E = weighted average of cell fill γ_T and γ_{SUB} (above and below water in the cell)

FIGURE 37 (continued)
Design Criteria for Cellular Cofferdams

14.	Friction angle of soil and steel.	$\delta = 2/3\ \phi'$
15.	Coefficient of friction between cell fill and rock.	λ = use 0.5 for smooth rock, for all other use $\tan\phi$
16.	Drained angle of shearing resistance of soil.	ϕ'
17.	Coefficient of interlock friction.	f = 0.3
18.	Horizontal effect stress on a vertial plane.	p' = (see pressure diagram for subscript)
19.	Horizontal effect force on a vertical plane.	P' = (see pressure diagram for subscipt)

FIGURE 37 (continued)
Design Criteria for Cellular Cofferdams

DESIGN METHODS

COFFERDAM ON ROCK - WITH BERM

1. Factor of safety against sliding on Base

$$F_S = \frac{W\lambda}{P_{WO} + P'_A - P_p} \geq 1.25 \text{ (TEMPORARY) TO } 1.5 \text{ (PERMANENT)}$$

2. Factor of safety against overturning, F_O

$$F_O = \frac{M_R}{M_O} \geq 3 \text{ TO } 3.5$$

3. Factor of safety against excessive interlock tension, F_i

$$F_i = \frac{T_u}{T} \geq 1.5 \text{ TO } 2.0$$

4. Factor of safety against vertical shear on centerline, F_{VS} (Terzaghi)

$$F_{VS} = 2/3 \frac{B}{M_O} \left[P'_{\mathfrak{C}} \tan\phi + (P'_i - P_p) f\right] \geq \begin{matrix} 1.25 \text{ (TEMPORARY WALL)} \\ 1.50 \text{ (PERMANENT WALL)} \end{matrix}$$

Where $P'_{\mathfrak{C}}$ is calculated using the effective stress diagram for the Center Plane of cell, and equals the area efgh with K = 0.5 to 0.6; and P'_i is calculated using the effective stress diagram of Inboard Sheeting, and equals area ab'c'd with K = 0.4.

5. Factor of safety against tilting, F_t

$$F_t = \frac{1}{M_O}\frac{1}{6}\, \gamma_E B^2 H \left(3\tan^2\phi - \frac{B}{H}\tan^3\phi + \frac{3KfH}{B}\right) \geq \begin{matrix} 1.25 \text{ (TEMPORARY)} \\ 1.50 \text{ (PERMANENT)} \end{matrix}$$

$$\text{FOR } K = \tan^2(45 - \phi/2)$$

6. Factor of safety against shear at cell fill, sheet pile interface, F_{sf}

$$F_{sf} = \frac{B}{M_O}\left[\left(P'_O + P'_A + \frac{P'_{\mathfrak{C}}}{L}\right)\tan\delta + P'_{\mathfrak{C}} f \frac{B}{L}\right] \geq \begin{matrix} 1.25 \text{ (TEMPORARY)} \\ 1.50 \text{ (PERMANENT)} \end{matrix}$$

Where P_O' is calculated using the effective stress diagram for Outboard Sheeting, and is equivalent to area jkl with K = 0.7 to 1.0.

7. Select value of B which satisfies all requirements.

COFFERDAM ON ROCK - WITHOUT BERM

Follow design Steps 1 through 7 as above for cofferdam with berm.

8. Put P_P = 0 in all equations to compute M_O and factor of safety.
9. In computing F_{VS}, P'_i is calculated using the stress diagram for Inboard Sheeting, and equals area ab'c'd with K = 0.4.

FIGURE 37 (Continued)
Design Criteria for Cellular Cofferdams

$$\text{WALL} = 1/2\, K_o\, \gamma_E\, D_1^{\,2}\, \tan\delta \times \text{PERIMETER (NOTE: } P_p = 0\text{), AND } Q_p = \frac{M_o}{3B\left(1 + \dfrac{B}{4L}\right)}$$

[retrieve FIGURE 37: (continued) Design Criteria for Cellular Cofferdams]

COFFERDAM ON SOFT TO MEDIUM STIFF CLAY

20. Design procedures same as for cofferdams on deep sand, with modifications as per following steps. Penetration to avoid piping is usually not important.

21. Factor of Safety for stability against bearing capacity failure, F_{bc}

F_{bc} from Step 12 ≥ 3

22. Because of internal instability due to settlement of compressible foundation, factor of safety against vertical stress on centerline F_{vs} from Step 4 should be

$$F_{VS} = \frac{P_{\mathfrak{C}}}{M_O} \times \frac{R f B}{L} \times \frac{(L+0.25B)}{(L+0.5B)} \geq \begin{cases} 1.25 \text{ (TEMPORARY)} \\ 1.50 \text{ (PERMANENT)} \end{cases}$$

Investigate overall stability of cofferdam with respect to sliding along a curved surface below the bottom of the sheeting by slope stability analysis from DM-7.1 CHAPTER 7.

23. Investigate and evaluate seams of pervious sand within the clay deposit which could develop excessive uplift pressure below the base of the cofferdam.

24. Evaluate penetration of outboard sheeting to avoid pull-out as per Step 19.

FIGURE 37 (continued)
Design Criteria for Cellular Cofferdams

(1) Sand Base. For cell walls on sand, penetration of sheeting must be sufficient to avoid piping at interior toe of wall and to prevent pullout of outboard sheeting.

(2) Clay Base. For cofferdams on clay, penetration of outboard sheeting usually is controlled by the pullout requirement and piping is not critical.

(3) Bearing Capacity. For cofferdams on either clay or sand, check the bearing capacity at the inboard toe by methods of Chapter 4.

c. Cell Deformations. The maximum bulging of cells occurs at about 1/4 of the height above the base of the cofferdam and the cells tilt about 0.02 to 0.03 radians due to the difference in lateral loads on the outboard and inboard faces. Deflections under the lateral overturning loads are a function of the dimensions, the foundation support, and the properties of the cell fill (see Reference 19, Field Study of Cellular Cofferdams, by Brown).

3. CELL FILL. Clean, coarse-grained, free-draining soils are preferred for cell fill. They may be placed hydraulically or dumped through water without compaction or special drainage.

a. Materials. Clean granular fill materials should be used in large and critical cells. Every alternative should be studied before accepting fine-grained backfill. These soils produce high bursting pressures and minimum cell rigidity. Their use may necessitate interior berms, increased cell width, or possibly consolidation by sand drains or pumping within the cell. All soft material trapped within the cells must be removed before filling.

b. Drainage. Weep holes should be installed on inboard sheeting to the cell fill. For critical cells and marginal fill material, supplementary drainage by wellpoints, or wells within cells have been used to increase cell stability.

c. Retardation of Corrosion. When cofferdams are used as permanent structures, (especially in brackish or seawater, severe corrosion occurs from top of the the splash zone to a point just below mean low water level. use Protective coating, corrosion resistant steel and/or cathodic protection in these areas.

REFERENCES

1. Canadian Geotechnical Society, Excavations and Retaining Structures, Canadian Foundation Engineering Manual, Part 4, 1978.

2. Caquot, A., and Kerisel, F., Tables for the Calculation of Passive Pressure, Active Pressure and Bearing Capacity of Foundations, Gauthier-Villars, Paris.

3. Gray, H., Contribution to the Analysis of Seepage Effects in Backfills, Geotechnique, 1958.

4. Richart, F.E., and Schmertmann, F., The Effects of Seepage on the Stability of Sea Walls, Proceedings, First Conference on Coastal Engineering, University of Florida, Gainesville, FL.

5. Based on work by Terzaghi, K., Anchored Bulkheads, Transaction, ASCE, Paper No. 2720, Vol. 119, 1954.

6. Goldberg, D.T., Jaworski, W.E., and Gordon, M.D., Lateral Support Systems and Underpinning, Vol. I, Design and Construction (Summary), FHWA-RD-75, Federal Highway Administration, 1976.

7. Ingold, T.S., Retaining Wall Performance During Backfilling, Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, GT5, 1979.

8. Seed, H.B. and Whitman, R.V., Design of Earth Retaining Structures for Dynamic Loads, Lateral Stresses in the Ground and Design of Earth Retaining structures, ASCE, Cornell University, 1970.

9. Terzaghi, K., and Peck, R.B., Soil Mechanics in Engineering Practice, John Wiley & Sons, Inc., New York, 1967.

10. Rowe, P.W., Anchored Sheet Pile Walls, Proceedings, Institution of Civil Engineers, London, January, 1952.

11. Tschebotarioff, G.P., Foundations, Retaining and Earth Structures, 2nd Edition, McGraw-Hill, New York, 1973.

12. United States Steel Corporation, Steel Sheet Piling Design Manual, United States Steel, Pittsburgh, PA., 1975.

13. Carson, A.C., Foundation Construction, McGraw-Hill Book Co., 1965.

14. Lee, K.L., Dean, B. and Vageron, J.M.J., Reinforced Earth Retaining Walls, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 99, No. SM10, 1973.

15. ASCE Proceedings, Symposium on Earth Reinforcement, ASCE Annual Convention, Pittsburg, PA., 1978.

16. Portland Cement Association, Concrete Crib Retaining Walls, Concrete Information No. St. 46, Chicago, IL., May, 1952.

17. Seelye, E.E., Foundations, Design and Practice, John Wiley & Sons, Inc., New York, New York, 1956.

18. Lacroix, Y., Esrig, M.I. and Luschem, U., Design, Construction and Performance of Cellular Cofferdams, Lateral Stresses in the Ground and Design of Earth Retaining Structures, ASCE, Cornell University, 1970.

19. Brown, P.P., Discussion of Paper by White, Chency and Duke, Field Study of Cellular Cofferdams, Transaction, ASCE, paper No. 3426, Vol. 128, Part 1, 1963.

20. Naval Facilities Engineering Command

Design Manuals (DM)

DM-2	Series on Structural Engineering
DM-25.04	Seawalls, Bulkheads, and Quaywalls

Copies of Guide Specifications and Design Manuals may be obtained from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120.

CHAPTER 4. SHALLOW FOUNDATIONS

Section 1. INTRODUCTION

1. SCOPE. This chapter presents criteria for the design of shallow foundations, methods of determining allowable bearing pressures, and treatment of problems in swelling and collapsing subsoils. For the majority of structures the design of footings is controlled by limiting settlements. (See RELATED CRITERIA below.) This chapter discusses permissible bearing pressures as limited by shear failure. Shallow foundations are of the following types; spread footings for isolated columns, combined footings for supporting the load from more than one structural unit, strip footings for walls, and mats or rafts beneath the entire building area. Also, included is guidance for footings subjected to uplift. Design of deep anchors for such footings is covered in DM-7.03, Chapter 3.

2. RELATED CRITERIA. See DM-7.01, Chapter 5 for determination of settlements of shallow foundations. See NAVFAC DM-2.02 for criteria for loads applied to foundations by various structures and structural design of foundations.

3. APPLICATIONS. Shallow foundations can be used where there is a suitable bearing stratum near the surface, no highly compressible layers below, and calculated settlements are acceptable. Where the bearing stratum at ground surface is underlain by weaker and more compressible materials, consider the use of deep foundations or piles. See Chapter 5.

Section 2. BEARING CAPACITY ANALYSIS

1. LIMITATIONS. Allowable bearing pressures for shallow foundations are limited by two considerations. The safety factor against ultimate shear failure must be adequate, and settlements under allowable bearing pressure should not exceed tolerable values. In most cases, settlement governs the foundation pressures. See DM-7.01, Chapter 5 for evaluation of settlements. For major structures, where relatively high foundation bearing pressures yield substantial economy, determine ultimate bearing capacity by detailed exploration, laboratory testing, and theoretical analysis. For small or temporary structures, estimate allowable bearing pressures from penetration tests, performance of nearby buildings, and presumptive bearing values; see Paragraphs 3 and 4.

2. THEORETICAL BEARING CAPACITY.

a. Ultimate Bearing Capacity. To analyze ultimate bearing capacity for various loading situations, see Figures 1 through 5. For these analyses the depth of foundation embedment is assumed to be less than the foundation width, and friction and adhesion on the foundation's vertical sides are neglected. In general, the analyses assume a rough footing base such as would occur with cast-in-place concrete.

Figures 1 through 5 present ultimate bearing capacity diagrams for the following cases:

(1) See Figure 1 (Reference 1, Influence of Roughness of Base and Ground Water Condition-on the Ultimate Bearing Capacity of Foundations, by Meyerhof) for shallow footings with concentric vertical load. Formulas shown assume groundwater at a depth below base of footing equal to or greater than the narrow dimension of the footing.

(2) Use Figure 2 (Reference 1) to determine groundwater effect on ultimate bearing capacity and the depth of failure zone. For cohesive soils, changes in groundwater level do not affect theoretical ultimate bearing capacity.

(3) Use Figure 3a (Reference 2, The Bearing Capacity of Foundations Under Eccentric and Inclined Loads, by Meyerhof) for inclined load on continuous horizontal footing and for inclined load on continuous inclined footing.

(4) Use Figure 3b for eccentric load on horizontal footing.

(5) Use Figures 4a; 4b (Reference 3, The Ultimate Bearing Capacity of Foundations on Slopes, by Meyerhof) for shallow footing with concentric vertical load placed on a slope or near top of slope.

(6) Use Figure 5 (Reference 4, The Bearing Capacity of Footings on a Two-Layer Cohesive Subsoil, by Button) for shallow footing with concentric vertical load on two layered cohesive soil.

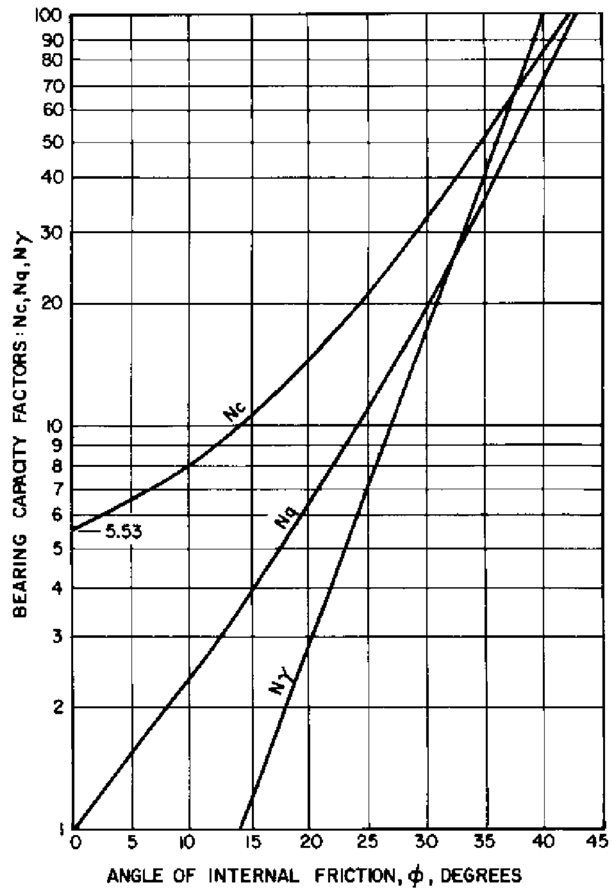
These diagrams assume general shear failure which normally occurs in dense and relatively incompressible soils. This type of failure is usually sudden and catastrophic; it is characterized by the existence of a well-defined failure pattern. In contrast, in loose or relatively compressible soils, punching or local shear failures may occur at lower bearing pressures. Punching or local shear failures are characterized by a poorly defined failure surface, significant vertical compression below the footing and very little disturbance around the footing perimeter.

To approximate the local or punching shear failures, the bearing capacity factors should be calculated with reduced strength characteristics c* and [phi]* defined as:

c* - 0.67 c
[phi]* = tan.1- (0.67 tan [phi])

For more detailed and precise analysis, see Reference 5, Bearing Capacity of Shallow Foundations, by Vesic.

b. Allowable Bearing Capacity. To obtain allowable bearing capacity, use a safety factor of 3 for dead load plus maximum live load. When part of the live loads are temporary (earthquake, wind, snow, etc.) use a safety factor of 2. Include in design dead load the effective weight of footing and soil directly above footing. See Figures 6 and 7 for examples of allowable bearing capacity calculations.



ULTIMATE BEARING CAPACITY = q_{ult}

CONTINUOUS FOOTING; GENERAL CASE

$q_{ult} = q' + q''$

q' = PORTION OF BEARING CAPACITY ASSUMING WEIGHTLESS FOUNDATION SOIL

q'' = PORTION OF BEARING CAPACITY FROM WEIGHT OF FOUNDATION SOILS

$q' = cN_c + \gamma D N_q$

$q'' = \gamma \frac{B}{2} N_\gamma$

$q_{ult} = cN_c + \gamma D N_q + \frac{\gamma B}{2} N_\gamma$

SQUARE OR RECTANGULAR FOOTING

$q_{ult} = cN_c\,(1 + .3\frac{B}{L}) + \gamma D N_q + 0.4\gamma B N_\gamma$

CIRCULAR FOOTING: R = B/2

$q_{ult} = 1.3\,cN_c + \gamma D N_q + 0.6\gamma R N_\gamma$

FOR COHESIONLESS FOUNDATION SOILS (c = 0)

CONTINUOUS FOOTING:

$q_{ult} = \gamma D N_q + \frac{\gamma B}{2} N_\gamma$

SQUARE OR RECTANGULAR FOOTING:

$q_{ult} = \gamma D N_q + 0.4\gamma B N_\gamma$

CIRCULAR FOOTING:

$q_{ult} = \gamma D N_q + 0.6\gamma R N_\gamma$

FOR COHESIVE FOUNDATION SOILS ($\phi = 0$)

CONTINUOUS FOOTING:

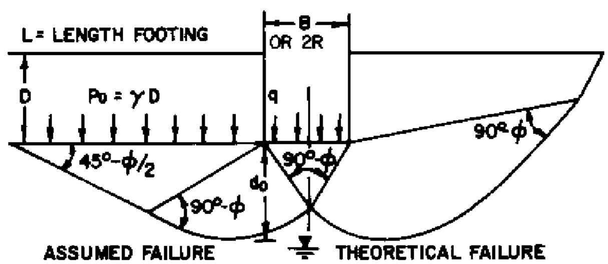
$q_{ult} = cN_c + \gamma D$

SQUARE OR RECTANGULAR FOOTING:

$q_{ult} = cN_c\,(1 + .3\frac{B}{L}) + \gamma D$

CIRCULAR FOOTING:

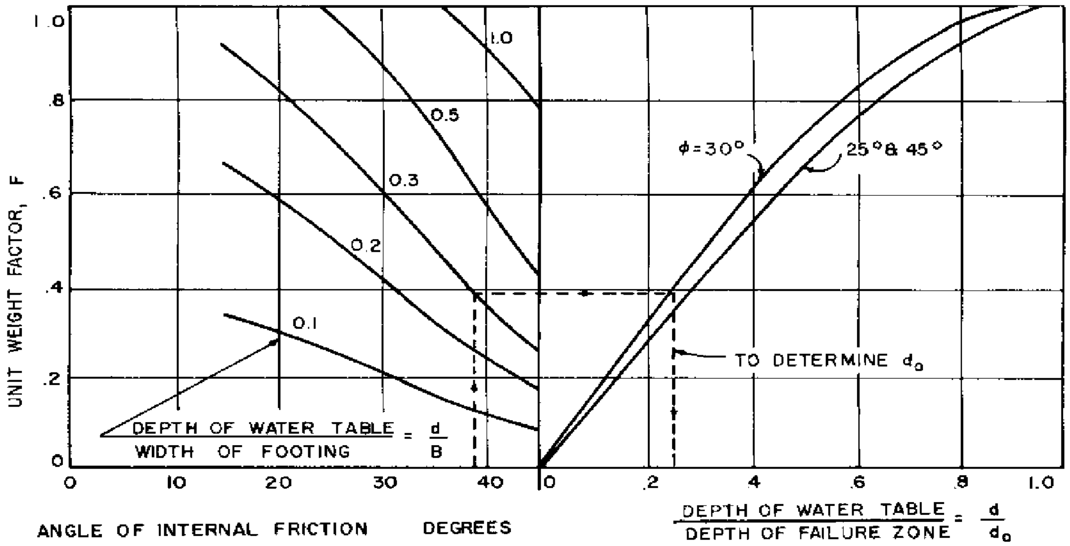
$q_{ult} = 1.3\,cN_c + \gamma D$

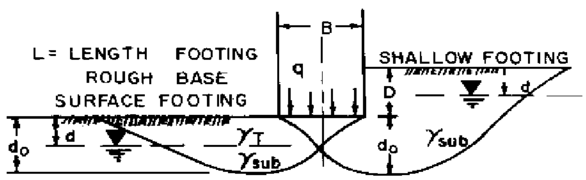


ASSUMED CONDITIONS:

1. $D \leq B$
2. SOIL IS UNIFORM TO DEPTH $d_o > B$.
3. WATER LEVEL LOWER THAN d_o BELOW BASE OF FOOTING.
4. VERTICAL LOAD CONCENTRIC.
5. FRACTION AND ADHESION ON VERTICAL SIDES OF FOOTING ARE NEGLECTED.
6. FOUNDATION SOIL WITH PROPERTIES c, ϕ, γ

FIGURE 1
Ultimate Bearing Capacity of Shallow Footings With Concentric Loads





ASSUMED CONDITIONS:

1. GROUNDWATER LEVEL IS HORIZONTAL
2. PRESENCE OF GROUNDWATER HAS NO EFFECT ON COHESIVE SOIL WITH $\phi = 0$.

CONTINUOUS FOOTING:

SURFACE FOOTING: D = 0

$$q_{ult} = cN_c + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})] \frac{B}{2} N_\gamma$$

SHALLOW FOOTING: D ≤ B

IF $d \leq D$

$$q_{ult} = cN_c + [\gamma_{sub} D + (\gamma_T - \gamma_{sub}) d] N_q + 0.5 \gamma_{sub} B N_\gamma$$

IF $D < d \leq (D + d_o)$

$$q_{ult} = cN_c + \gamma_T D N_q + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})] \frac{B}{2} N_\gamma$$

VALUES OF BEARING CAPACITY FACTORS N_c N_q AND N_γ ARE SHOWN IN FIGURE 1.

RECTANGULAR FOOTING:

SURFACE FOOTING: D = 0

$$q_{ult} = cN_c(1 + .3\frac{B}{L}) + [\gamma_{sub} + F(\gamma - \gamma_{sub})] \, 0.4 B N_\gamma$$

SHALLOW FOOTING: D ≤ B, IF d ≤ D

$$q_{ult} = cN_c(1 + .3\frac{B}{L}) + [\gamma_{sub} D + (\gamma_T - \gamma_{sub}) d] N_q + 0.4 \gamma_{sub} B N_\gamma$$

IF $D < d \leq (D + d_o)$

$$q_{ult} = cN_c(1 + .3\frac{B}{L}) + \gamma_T D N_q + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})] \, 0.4 B N_\gamma$$

CIRCULAR FOOTING: RADIUS = R = B/2

SURFACE FOOTING: D = 0

$$q_{ult} = 1.3 cN_c + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})] \, 0.6 R N_\gamma$$

SHALLOW FOOTING: D ≤ 2R, IF d ≤ D

$$q_{ult} = 1.3 cN_c + [\gamma_{sub} D + (\gamma_T - \gamma_{sub}) d] N_q + [0.6 \gamma_{sub} R N_\gamma]$$

IF $D < d \leq (D + d_o)$

$$q_{ult} = 1.3 cN_c + \gamma_T D N_q + [\gamma_{sub} + F(\gamma_T - \gamma_{sub})] \, 0.6 R N_\gamma$$

FIGURE 2
Ultimate Bearing Capacity With Groundwater Effect

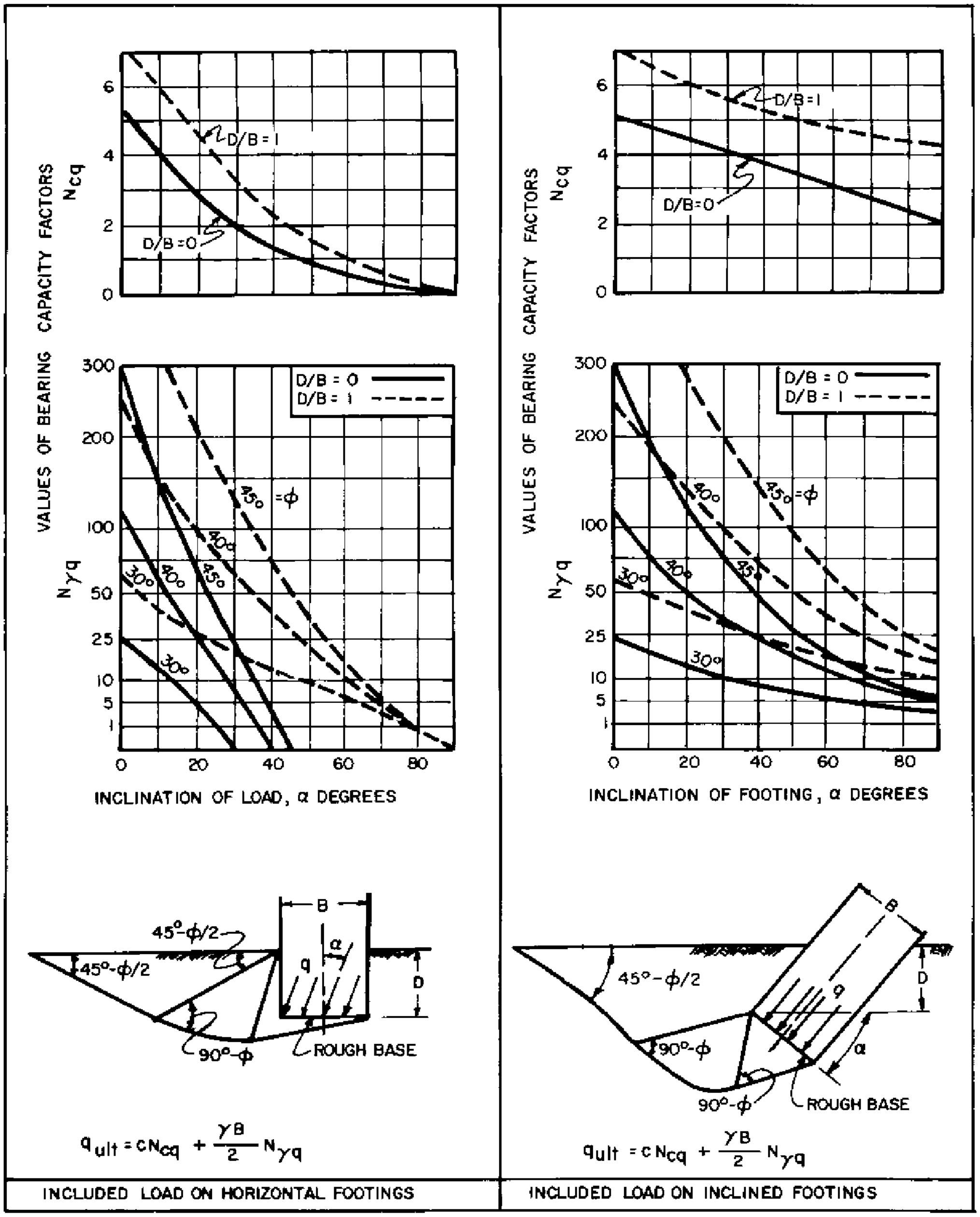
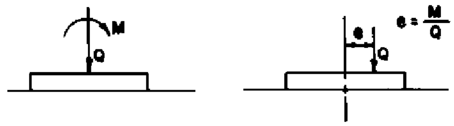
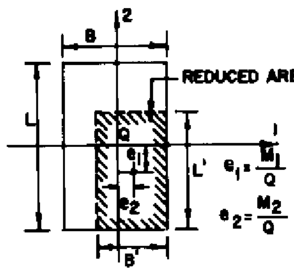


FIGURE 3a
Ultimate Bearing Capacity of Continuous Footings With Inclined Load



(A) EQUIVALENT LOADINGS

Resultant force acts at the centroid of the reduced area.



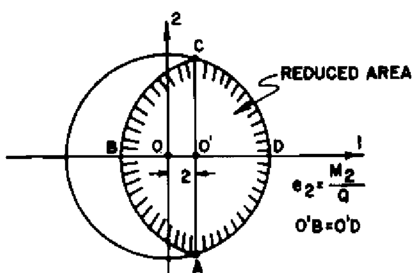
(B) REDUCED AREA-RECTANGULAR FOOTING

For rectangular footings reduce dimension as follows:

$L' = L - 2e_1 \qquad e_1 = \frac{M_1}{Q}$

$B' = B - 2e_2 \qquad e_2 = \frac{M_2}{Q}$

For a circular footing of radius R, the effective area A'_e = 2 x (area of circular segment ADC), consider A'_e to be a rectangle with $L'/B' = \frac{AC}{BD}$



(C) REDUCED AREA-CIRCULAR FOOTING

$e = \frac{M}{Q}$

$A'_e = 2S = B'L'$

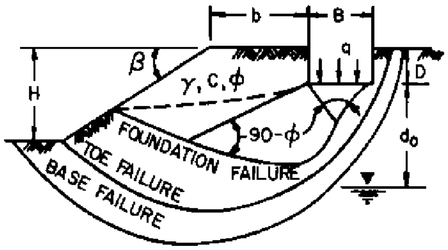
$L' = \left(2S\sqrt{\frac{R+e_2}{R-e_2}}\right)^{1/2}$

$B' = L'\sqrt{\frac{R-e_2}{R+e_2}}$

$S = \frac{\pi R^2}{2} - \left[e_2\sqrt{R^2-e_2^2} + R^2 \sin^{-1}\left(\frac{e_2}{R}\right)\right]$

FIGURE 3b
Eccentrically Loaded Footings

CASE I: CONTINUOUS FOOTING AT TOP OF SLOPE



Water at $d_o \geq B$

$$q_{ult} = cN_{cq} + \gamma_T \frac{B}{2} N_{\gamma q} \quad (1)$$

Water at Ground Surface

$$q_{ult} = cN_{cq} + \gamma_{sub} \frac{B}{2} N_{\gamma q} \quad (2)$$

If $B \leq H$:

Obtain N_{cq} from Figure 4b for Case I with $N_o = 0$.
Interpolate for values of $0 < D/B < 1$

Interpolate q_{ult} between EQ (1) and (2) for water at intermediate level between ground surface and $d_o = B$.

If $B > H$:

Obtain N_{cq} from Figure 4b for Case I with stability number

$$N_o = \frac{\gamma H}{C}.$$

Interpolate for values $0 < D/B < 1$ for $0 < N_o < 1$. If $N_o \geq 1$, stability of slope controls ultimate bearing pressure.

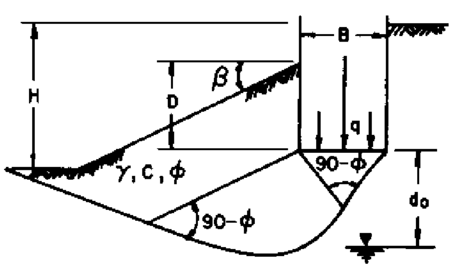
Interpolate q_{ult} between EQ (1) and (2) for water at intermediate level between ground surface and $d_o = B$. For water at ground surface and sudden drawdown: substitute ϕ' for ϕ in EC (2)

$$\phi' = \tan^{-1}\left(\frac{\gamma_{sub}}{\gamma_T} \tan \phi\right)$$

Cohesive soil ($\phi = 0$)
Substitute in EQ (1) and (2) D for $B/2$ and $N_{\gamma q} = 1$.
Rectangular, square or circular footing:

$$q_{ult} = \left[q_{ult} \text{ for continuous footing as given above}\right] \times \left[\frac{q_{ult} \text{ for finite footing}}{q_{ult} \text{ for continuous footing}}\right] \text{ from Fig. 1}$$

CASE II: CONTINUOUS FOOTINGS ON SLOPE



Same criteria as for Case I except that N_{cq} and $N_{\gamma q}$ are obtained from diagrams for Case II

FIGURE 4a
Ultimate Bearing Capacity For Shallow Footing Placed on or Near a Slope

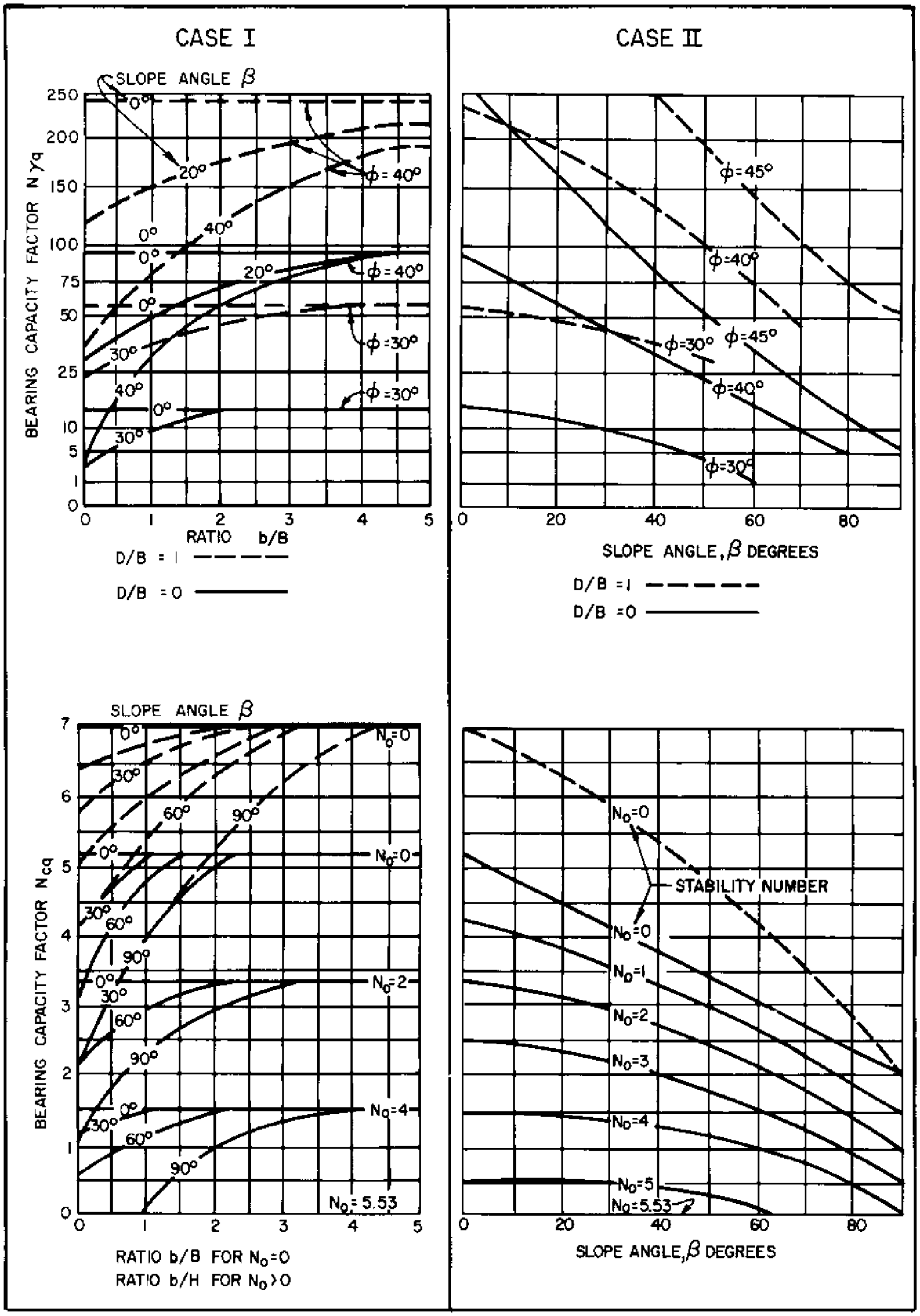


FIGURE 4b
Bearing Capacity Factors for Shallow Footing Placed on or Near a Slope

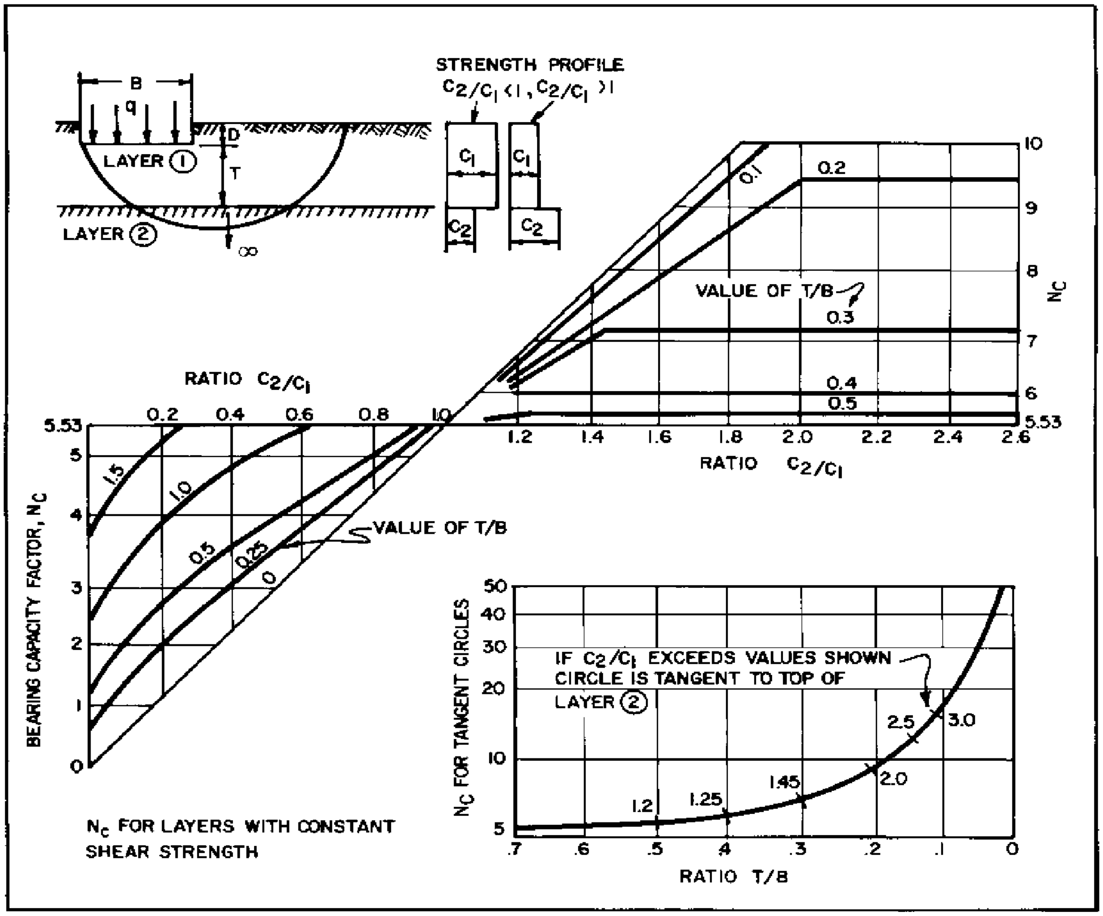
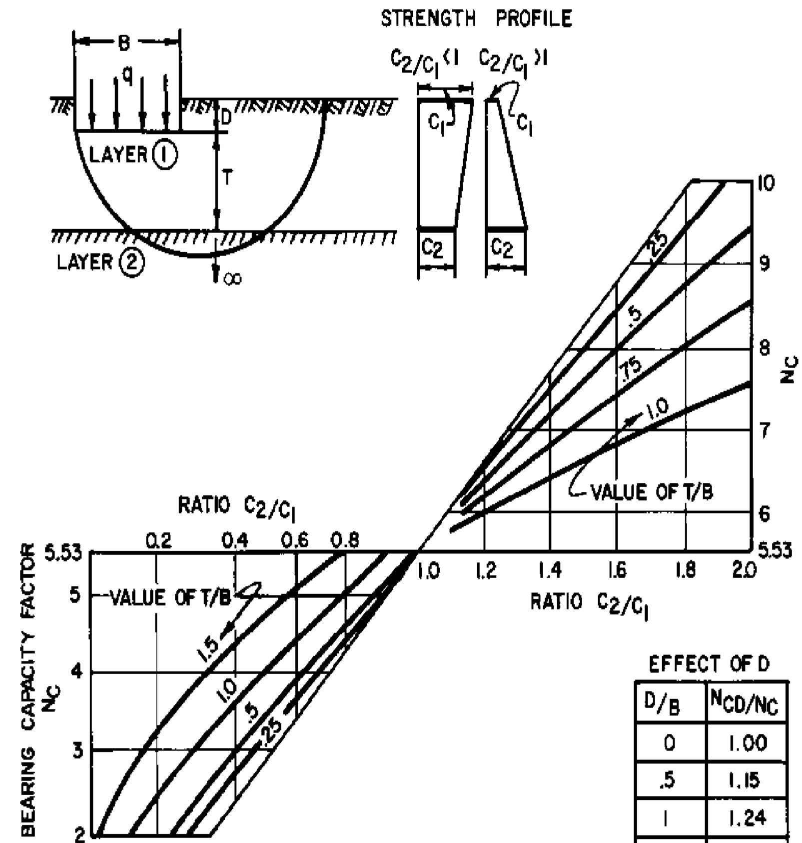


FIGURE 5
Ultimate Bearing Capacity of Two Layer Cohesive Soil (ϕ=0)



EFFECT OF D

D/B	N_{CD}/N_C
0	1.00
.5	1.15
1	1.24
2	1.36
3	1.43
4	1.46

N_c FOR FOUNDATION WITH UPPER LAYER OF VARIABLE SHEAR STRENGTH

LEGEND

D = DEPTH OF EMBEDMENT OF FOOTING

N_C = BEARING CAPACITY FACTOR FOR CONTINUOUS FOOTING WITH D=0

N_{CD} = FACTOR FOR CONTINUOUS FOOTING WITH D > 0

N_{CR} = FACTOR FOR RECTANGULAR FOOTING WITH D=0

CONTINUOUS FOOTING

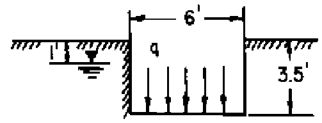
$q_{ult} = c_1 N_{CD} + \gamma D$

N_{CD}/N_C FROM TABLE ABOVE

RECTANGULAR FOOTING

$N_{CR} = N_{CD}\left[1+0.2\left(\frac{B}{L}\right)\right], q_{ult} = c_1 N_{CR} + \gamma D$

FIGURE 5 (continued)
Ultimate Bearing Capacity of Two Layer Cohesive Soil (Ø=0)



C = 2.0 KSF $\phi = 0$

γ_T = 130 PCF

N_C (FROM FIGURE 1) = 5.53

$q_{ult} = cN_c + \gamma D$ { γD = EFFECTIVE SURCHARGE PRESSURE AT LEVEL OF BASE OF FOOTING.

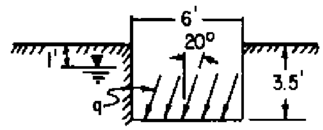
$q_{ult} = 2(5.53) + [.13(1) + (.13 - .0625)(2.5)] = 11.4$ KSF

FOR FACTOR OF SAFETY, $F_S = 3.0$

$$q_{all} = \frac{q_{ult}}{3.0} = \frac{11.4}{3.0} = 3.8 \text{ KSF}$$

DEAD LOADS TO INCLUDE EFFECTIVE WEIGHT OF FOOTING WITH ASSUMED GROUND WATER LEVEL.

(A) CONTINUOUS FOOTING WITH VERTICAL LOAD



C = 2.0 KSF $\phi = 0$

γ_T = 130 PCF

$D/B = \frac{3.5}{6} = 0.58$

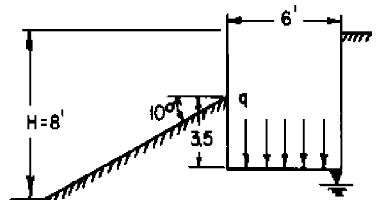
N_{cq} (FROM FIGURE 3) = 3.8, $N_{\gamma q} = 0$

$q_{ult} = cN_{cq} + \frac{\gamma B}{2} N_{\gamma q} = 2(3.8) + 0 = 7.6$ KSF

$F_S = 3.0$

$$q_{all} = \frac{7.6}{3.0} = 2.5 \text{ KSF}$$

(B) CONTINUOUS FOOTING WITH INCLINED LOAD



C=2.0 KSF $\phi = 0$ γ_T = 130 PCF

STABILITY NUMBER, $N_0 = \frac{\gamma_T H}{C} = 0.52$

FOR D/B=1, N_{cq} = 6.75 } FROM FIGURE 4
FOR D/B=0, N_{cq} = 4.3

q_{ult} IS INTERPOLATED BETWEEN VALUES FOR D/B=1 AND D/B=0

FOR D/B=1, $q_{ult} = cN_{cq} + \gamma D = 2(6.75) + .13(3.5) = 14.0$ KSF

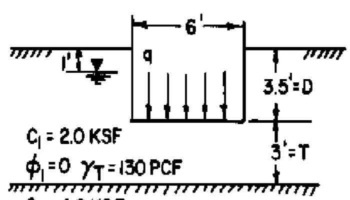
FOR D/B=0, $q_{ult} = 2(4.3) + .13(3.5) = 9.1$ KSF

FOR D/B=3.5/6=0.58, $q_{ult} = 9.1 + 0.58(4.9) = 11.9$ KSF

$F_S = 3.0$

$$q_{all} = \frac{11.9}{3.0} = 4.0 \text{ KSF}$$

(C) CONTINUOUS FOOTING PLACED ON SLOPE



C_1 = 2.0 KSF

$\phi_1 = 0$ γ_T = 130 PCF

C_2 = 4.0 KSF

$\phi_2 = 0$ γ_T = 130 PCF

$C_2/C_1 = 4/2 = 2$ T/B = 3/6 = 0.5

N_C (FROM FIGURE 5) = 5.8

FOR D/B = 3.5/6 ≅ 0.5, N_{CD}/N_C (FROM FIGURE 5) = 1.15

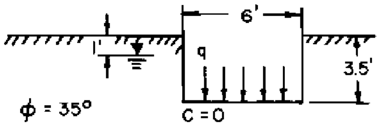
$q_{ult} = c_1 N_{CD} + \gamma D = 2(5.8)(1.15) + [.13(1) + (.13 - .0625)(2.5)] = 13.6$ KSF

$F_S = 3.0$

$$q_{all} = \frac{13.6}{3.0} = 4.5 \text{ KSF}$$

(D) CONTINUOUS FOOTING ON TWO-LAYER FOUNDATION STRATA

FIGURE 6
Examples of Computation of Allowable Bearing Capacity
Shallow Footings on Cohesive Soils



$\phi = 35°$ C = 0
$\gamma_T = 130$ PCF

N_q (FROM FIGURE 1) = 34
N_γ (FROM FIGURE 1) = 40 (APPROX.)

FROM FIGURE 2 AND $D \leq B$:

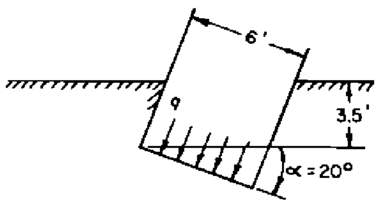
$q_{ult} = cN_c + [\gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d] N_q + 0.5 \gamma_{SUB} B N_\gamma$
$= 0 + [.068 \times 3.5 + (.13 - .068) 1.0] 34 + 0.5 \times .068 \times 6 \times 40$
$= 18.5$ KSF

$F_S = 3.0$

$q_{all} = \frac{18.5}{3.0} = 6.2$ KSF

DEAD LOAD TO INCLUDE EFFECTIVE WEIGHT OF FOOTING WITH ASSUMED GROUND WATER LEVEL.

(A) CONTINUOUS FOOTING WITH VERTICAL LOAD



$\phi = 35°$ C = 0
$\gamma_T = 130$ PCF

$N_{\gamma q}$ (FROM FIGURE 3) = 60
D/B = 3.5/6 = .58

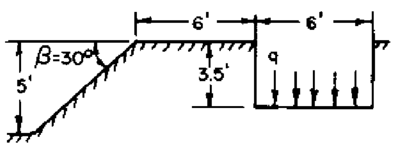
ASSUME GROUND WATER TABLE IS LOCATED AT GREAT DEPTH BELOW BASE OF FOOTING.

$q_{ult} = cN_{cq} + \gamma \frac{B}{2} N_{\gamma q} = 0 + .130 \times \frac{6}{2} \times 60 = 23.4$ KSF

$F_S = 3.0$

$q_{all} = \frac{23.4}{3.0} = 7.8$ KSP

(B) INCLINED CONTINUOUS FOOTING WITH INCLINED LOAD



$\phi = 35°$ C = 0
$\gamma_T = 130$ PCF

$\frac{b}{B} = \frac{6}{6} = 1.0$ $\frac{D}{B} = \frac{3.5}{6} = .58$

$N_{\gamma q}$ (FROM FIGURE 4)

$N_{\gamma q} = \frac{10+40+38+120}{8} = 26$

ASSUME GROUND WATER TABLE IS LOCATED AT GREAT DEPTH BELOW BASE OF FOOTING.

$q_{ult} = cN_{cq} + \gamma \frac{B}{2} N_{\gamma q} = 0 + .13 \times \frac{6}{2} \times 26 = 10.2$ KSF

$F_S = 3.0$

$q_{all} = \frac{10.2}{3.0} = 3.4$ KSF

(C) CONTINUOUS FOOTING PLACED BACK OF SLOPE

FIGURE 7
Examples of Computation of Allowable Bearing Capacity
Shallow Footings on Granular Soils

c. Soil Strength Parameters.

(1) Cohesive Soils. In the case of fine-grained soils which have low permeability, total stress strength parameters are used. Value of cohesion may be determined from laboratory unconfined compression tests, vane shear tests, or undrained triaxial tests. Shear strength correlations with standard penetration tests and cone penetration tests may also be used. (See DM-7.1, Chapter 1.)

(2) Granular Soils. In the case of coarse-grained soils which drain freely use the effective stress strength parameter ([phi]'). Field tests (e.g., standard penetration tests or cone penetration) are almost always used to estimate this strength.

(3) In the case where partial drainage may occur during construction (e.g., newly compacted fill) perform two analyses, one assuming drained, the other assuming undrained conditions, and design for the most conservative results.

3. PRESUMPTIVE BEARING PRESSURES. For preliminary estimates or when elaborate investigation of soil properties is not justified, use bearing pressure from Table 1.

a. Utilization. These load intensities are intended to provide a reasonable safety factor against ultimate failure and to avoid detrimental settlements of individual footings. Where differential settlements cannot be tolerated, exploration, testing and analysis should be performed. Presumptive bearing pressures must be used with caution and verified, if practicable, by performance of nearby structures.

b. Modifications of Presumptive Bearing Pressures. See Table 2 for variations in allowable bearing pressure depending on footing size and position. (See Reference 6, Foundation Analysis and Design, by Bowles for more detailed analyses of uplift resistance than shown in Table 2). Nominal bearing pressures may be unreliable for foundations on very soft to medium-stiff fine-grained soils or over a shallow groundwater table and should be checked by an estimate of theoretical bearing capacity. Where bearing strata are underlain by weaker and more compressible material, or where compressibility of subsoils is constant with depth, analyze consolidation settlement of the entire foundation (see DM-7.1, Chapter 5).

4. EMPIRICAL, ALLOWABLE BEARING PRESSURES. Allowable bearing pressures for foundation May be based upon the results of field tests such as the Standard Penetration Test (SPT) or Cone Penetration Test (CPT). These bearing pressures are based on maximum foundation settlements, but do not consider settlement effects due to the adjacent foundations. In the case of closely spaced foundations where the pressure beneath a footing is influenced by adjoining footings a detailed settlement analysis must be made.

TABLE 1
Presumptive Values of Allowable Bearing Pressures for Spread Foundations

Type of Bearing Material	Consistency In Place	Allowable Bearing Pressure Tons Per sq ft Range	Recommended Value for Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks).	Hard, sound rock	60 to 100	80.0
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks).	Medium hard sound rock	30 to 40	35.0
Sedimentary rock; hard cemented shales, siltstone, sandstone, limestone without cavities.	Medium hard sound rock	15 to 25	20.0
Weathered or broken bed rock of any kind except highly argillaceous rock (shale). RQD less than 25.	Soft rock	8 to 12	10.0
Compaction shale or other highly argillaceous rock in sound condition.	Soft rock	8 to 12	10.0
Well graded mixture of fine and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	8 to 12	10.0
Gravel, gravel-sand mixtures, boulder gravel mixtures (SW, SP, SW, SP)	Very compact	6 to 10	7.0
	Medium to compact	4 to 7	5.0
	Loose	2 to 6	3.0
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	4 to 6	4.0
	Medium to compact	2 to 4	3.0
	Loose	1 to 3	1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact	3 to 5	3.0
	Medium to compact	2 to 4	2.5
	Loose	1 to 2	1.5

TABLE 1 (continued)
Presumptive Values of Allowable Bearing Pressures for Spread Foundations

Type of Bearing Material	Consistency In Place	Allowable Bearing Pressure Tons Per sq ft. Range	Recommended Value for Use
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard	3 to 6	4.0
	Medium to stiff	1 to 3	2.0
	Soft	.5 to 1	0.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine Sand	Very stiff to hard	2 to 4	3.0
	medium to stiff	1 to 3	1.5
	Soft	.5 to 1	0.5

Notes:

1. Variations of allowable bearing pressure for size, depth and arrangement of footings are given in Table 2.

2. Compacted fill, placed with control of moisture, density, and lift thickness, has allowable bearing pressure of equivalent natural soil.

3. Allowable bearing pressure on compressible fine grained soils is generally limited by considerations of overall settlement of structure.

4. Allowable bearing pressure on organic soils or uncompacted fills is determined by investigation of individual case.

5. If tabulated recommended value for rock exceeds unconfined compressive strength of intact specimen, allowable pressures equals unconfined compressive strength.

TABLE 2

Selection of Allowable Bearing Pressures for Spread Foundations

1. For preliminary analysis or in the absence of strength tests of foundation soil, design and proportion shallow foundations to distribute their loads using presumptive values of allowable bearing pressure given in Table 1. Modify the nominal value of allowable bearing pressure for special conditions in accordance with the following items.

2. The maximum bearing pressure beneath the footing produced by eccentric loads that include dead plus normal live load plus permanent lateral loads, shall not exceed the nominal bearing pressure of Table 1.

3. Bearing pressures up to one-third in excess of the nominal bearing values are permitted for transient live load from wind or earthquake. If overload from wind or earthquake exceeds one-third of nominal bearing pressures, increase allowable bearing pressures by one-third of nominal value.

4. Extend footings on soft rock or on any soil to a minimum depth of 18 inches below adjacent ground surface or surface of adjacent floor bearing on soil, whichever elevation is the lowest.

5. For footings on soft rock or on coarse-grained soil, increase allowable bearing pressures by 5 percent of the nominal values for each foot of depth below the minimum depth specified in 4.

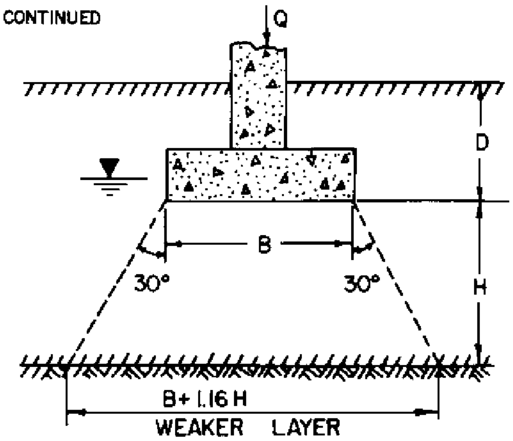
6. Apply the nominal bearing pressures of the three categories of hard or medium hard rock shown on Table 1 where base of foundation lies on rock surface. Where the foundation extends below the rock surface increase the allowable bearing pressure by 10 percent of the nominal values for each additional foot of depth extending below the surface.

7. For footing smaller than 3 feet in least lateral dimension, the allowable bearing pressure shall be one-third of the nominal bearing pressure multiplied by the least lateral dimension in feet.

8. Where the bearing stratum is underlain by a weaker material determine the allowable bearing pressure as follows:

TABLE 2 (continued)
Selection of Allowable Bearing Pressures for Spread Foundations

8. CONTINUED



Q = applied load, not including weight of foundation itself.

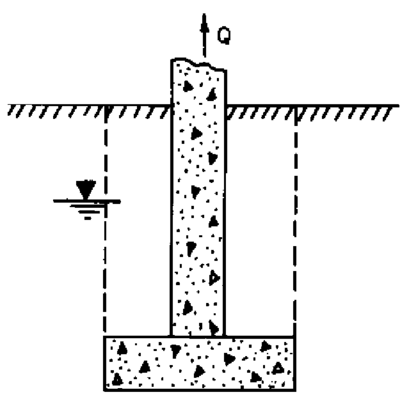
L = length of foundation.

(B + 1.16H) (L + 1.16H) = area stressed in weaker layer.

$$\frac{Q}{(B + 1.16H)(L + 1.16H)} \leq \text{nominal value of allowable bearing pressure.}$$

Area stressed in weaker layer shall not extend beyond intersection of 30° planes extending downward from adjacent foundations.

9. Where the footing is subjected to a sustained uplift force, compute ultimate resistance to uplift as follows:



Q = applied uplift load.

W = total effective weight of soil and concrete located within prism bounded by vertical lines at base of foundation. Use total unit weights above water table and buoyant unit weights below.

$$\text{Safety Factor} = \frac{W}{Q} \geq 2$$

(This is a conservative procedure; see text for reference on more detailed analyses procedures.)

a. Standard Penetration Test. Relationships are presented in Reference 7, Foundation Engineering, by Peck, Hanson and Thornburn, for allowable bearing values in terms of standard penetration resistance and for limiting settlement. When SPT tests are available, use the correlation in DM-7.1, Chapter 2 to determine relative density and Figure 6, DM-7.1, Chapter 3 to estimate [phi] values. Use Figure 1 to compute ultimate bearing pressure.

b. Cone Penetration Test. The results of CPT may be used directly to compute allowable bearing pressure for coarse-grained soils. See Figure 8 (Reference 8, Shallow Foundations, by the Canadian Geotechnical Society).

c. Bearing Capacity From Pressuremeter. If pressuremeter is used to determine in situ soil characteristics, bearing capacity can be computed from these test results. (See Reference 8.)

Section 3. SPREAD FOOTING DESIGN CONSIDERATIONS

1. FOUNDATION DEPTH. In general footings should be carried below:

(a) The depth of frost penetration;

(b) Zones of high volume change due to moisture fluctuations;

(c) Organic materials;

(d) Disturbed upper soils;

(e) Uncontrolled fills;

(f) Scour depths in rivers and streams.

(g) Zones of collapse-susceptible soils.

2. ALTERNATIVE FOUNDATION METHODS - Light Structures. Light structures may be supported by other types of shallow foundation treatment such as: (a) deep perimeter wall footings; (b) overexcavation and compaction in footing lines; (c) mat design with thickened edge; (d) preloading surcharge.

3. PROPORTIONING INDIVIDUAL FOOTINGS. Where significant compression will not occur in strata below a depth equal to the distance between footings, individual footings should be proportioned to give equal settlements, using formulas from DM-7.1, Chapter 5. See Figure 9 for an example.

4. CORROSION PROTECTION. Foundation design should consider potentially detrimental substances in soils, such as chlorides and sulphates, with appropriate protection for reinforcement, concrete and metal piping. If the analysis indicates sulphate concentration to be more than 0.5% in the soil or more than 1200 parts per million in the groundwater, the use of a sulphate resisting cement such as Type V Portland cement should be considered. In additions, other protection such as lower water-cement ratio, bituminous coating, etc. may be required depending upon the sulphate concentration. See Reference 9, Sulphates in soils and Groundwaters, BRS Digest, for guidance.

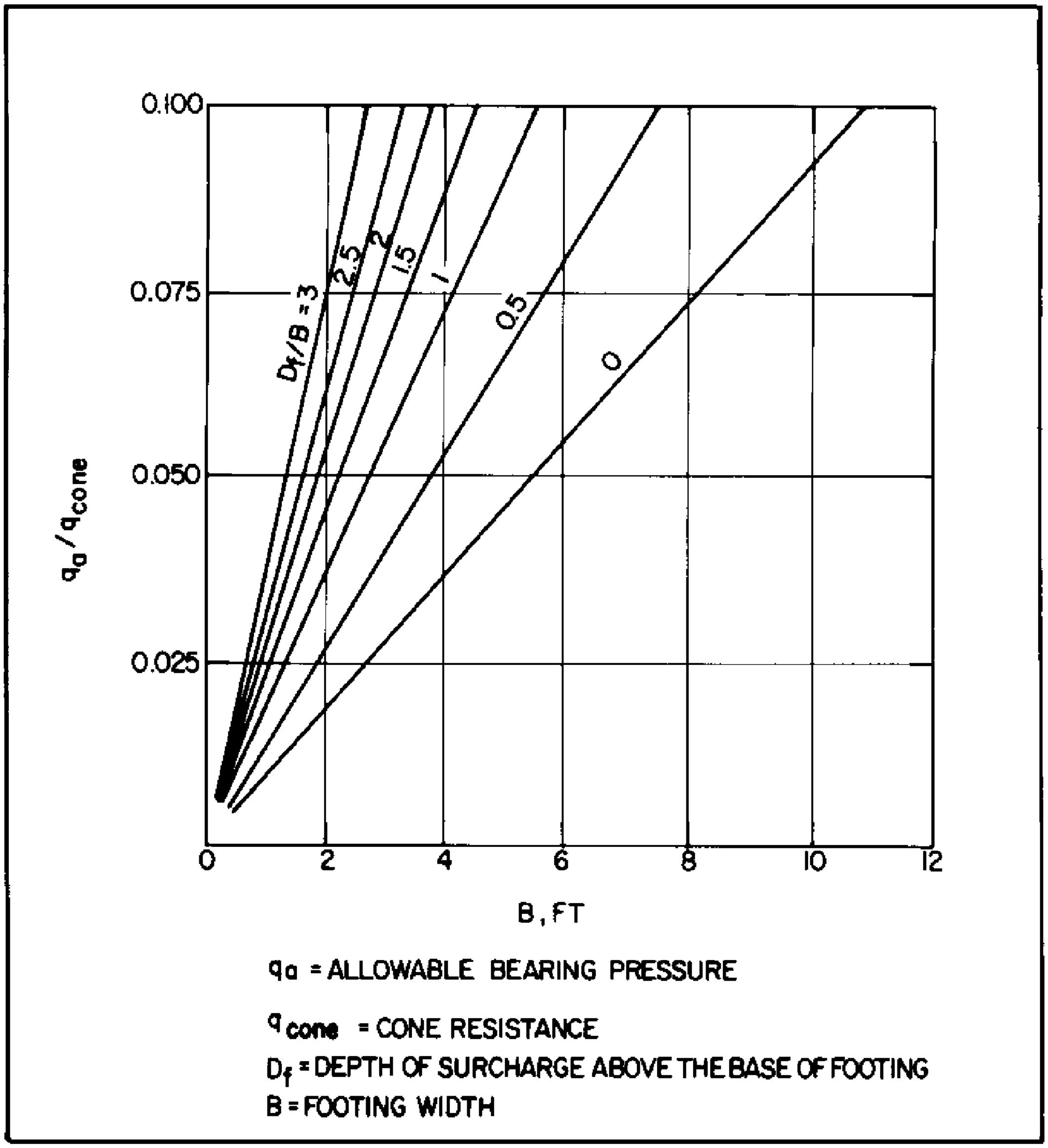
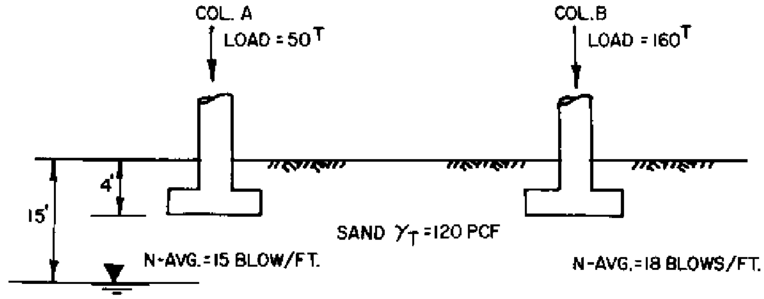


FIGURE 8
Allowable Bearing Pressure for Sand From Static Cone Penetration Tests

EXAMPLE



Column load A = 50 tons , Avg. N = 15 blows/ft.
Column load B = 160 tons , Avg. N = 18 blows/ft.
Soil: well graded sand (SW) , γ_T = 120 pcf

Column A

Assume square footing 5ft. x 5ft., B = 5ft.
Average overburden pressure at 6.5 ft. (D_f + B/2) below ground level:

P_o = 120 x 6.5 = 780 psf = 0.39 tsf

From Figure 3, DM-7.1, Chapter 2, D_r = 80%

From Figure 7, DM-7.1, Chapter 3, ϕ = 37.5°

a) Determine Bearing Capacity

From Figure 1, $q_{ult} = [120 \times 4 \times 45 + 0.4 \times 120 \times 5 \times 70] \dfrac{1}{2000} = 19.2$ tsf

$q_{ult\,(net)} = 19.2 - \dfrac{120 \times 4}{2000} \simeq 19$ tsf

Use F_s = 3, $\therefore q_{all} = \dfrac{19}{3} = 6.3$ tsf

Minimum required footing size: $\dfrac{50}{6.3} \approx$ 3ft. x 3ft. which is less than assumed size 5ft. x 5ft.

b) Check for settlement.

To limit settlement, assume a 5ft. x 5ft. footing with $q = \dfrac{50^T}{5\text{ft.} \times 5\text{ft.}} = 2$ tsf.

From Figure 6, DM-7.1, Chapter 5 K_{v1} = 255 tons/ft^3

$\Delta H = \dfrac{4 \times 2 \times 5^2}{255 \times (5 + 1)^2} \times 12 = 0.26$ inches

Column B

Assume 8ft. x 8ft. square footing
Average overburden pressure at 8ft. = (D_f + B/2) below ground level.

$P_o = 120 \times 8 \times \dfrac{1}{2000} = 0.48$ tsf

From Figure 3, DM-7.1, Chapter 2, D_r = 87%

From Figure 7, DM-7.1, Chapter 3, ϕ = 39°

a) Determine Bearing Capacity

From Figure 1, $q_{ult} = [120 \times 4 \times 58 + 0.4 \times 120 \times 8 \times 96] \dfrac{1}{2000} = 32.3$ tsf

$q_{ult\,(net)} = 32.3 - \dfrac{120 \times 4}{2000} \approx 32$ tsf

Use F_s = 3.0 $\therefore q_{all} = \dfrac{32}{3} = 10.7$ tsf

FIGURE 9
Example of Proportioning Footing Size to Equalize Settlements

Minimum required footing size: $\frac{160}{10.7}$ = 3.9 ft. x 3.9 ft.

b) Footing size required for settlement equal to that of Column A.

From Figure 6, DM-7.1, Chapter 5, K_{v1} = 290 tons/ft.^{3}

$$0.26 = \frac{4 \times 160 \times B^2}{290 \times B^2 \times (B+1)^2} \times 12$$

Or $B = \sqrt{\frac{4 \times 160 \times 12}{0.26 \times 290}} - 1 = 9.1 \gg 3.9$

Settlement Governs

Use 9.1 x 9.1 footing for Column B

FIGURE 9 (continued)
Example of Proportioning Footing Size to Equalize Settlements

Electrical corrosive properties of soil are important where metal structures such as pipe lines, etc. are buried underground. A resistivity survey of the site may be necessary to evaluate the need for cathodic protection.

Section 4. MAT AND CONTINUOUS BEAM FOUNDATIONS

1. APPLICATIONS. Depending on economic considerations mat foundations are generally appropriate if the sum of individual footing base areas exceeds about one-half the total foundation area; if the subsurface strata contain cavities or compressible lenses; if shallow shear strain settlements predominate and the mat would equalize differential settlements; or if resistance to hydrostatic uplift is required.

2. STABILITY AND SETTLEMENT REQUIREMENTS. As with other types of foundations, a mat foundation must have an ample factor of safety (see Section 2) against overall shear failure and it must not exhibit intolerable settlement (see DM-7.1, Chapter 5).

Since mat footings are simply large footings, the bearing capacity principles outlined in Sections 2 and 3 of this chapter are applicable. The ultimate bearing capacity of large mats on coarse-grained soils is usually very high and design is usually controlled by settlement (see DM-7.1, Chapter 5). For mats on cohesive soils, shear strength parameters for soils at depth must be determined for the proper evaluation of factor of safety against deepseated failure.

3. DESIGN PROCEDURES. A design method based on the theory for beams or plates on discreet elastic foundations (Reference 10, Beams on Elastic Foundation, by Hetenyi) has been recommended by ACI Committee 436 (Reference 11, Suggested Design Procedures for Combined Footings Mats) for design ofmat foundations. This analysis is suitable for foundations on coarse-grained soils.

a. Two-dimensional Problems. For walls or crane track footings or mat foundations subjected to plane strain, such as drydock walls and linear blocking loads, use the procedures of Table 3 and Figures 10 and 11 (Reference 10). Superpose shear, moment, and deflection produced by separate loads to obtain the effect of combined loads.

b. Three-dimensional Problems. For individual loads applied in irregular pattern to a roughly equi-dimensional mat, analyze stresses by methods of plates on elastic foundations. Use the procedures of Table 4 and Figure 12.

Superpose shear, moment, or deflection produced by separate loads to obtain the effect of combined loads.

TABLE 3

Definitions and Procedures, Analysis of Beams on Elastic Foundation

Definitions:

K_{v1} = Modulus of subgrade reaction for a 1 sq ft bearing plate.

K_b = Modulus of subgrade reaction for beam of width b, $K_b = (K_{v1})/b$

y = Deflection of beam at a point.

p = Pressure intensity on the subgrade at a point, $p = y(K_b)$

b = Width of beam at contact surface

I = Moment of inertia of beam

E = Modulus of elasticity of beam material

ℓ = Beam length

λ = Characteristics of the system of beam and supporting soil =

$$\lambda = \sqrt[4]{\frac{K_b b}{4\ EI}}$$

Procedure for Analysis:

1. Determine E and establish K_{v1} from Figure 6 in DM-7.01, Chapter 5 or from plate bearing tests.

2. Determine depth of beam from shear requirements at critical section and width from allowable bearing pressure. Compute characteristic λ of beam and supporting soil.

3. Classify beams in accordance with relative stiffness into the following three groups. Analysis procedure differs with each group.

   Group 1 - Short beams: $\lambda\ell < \pi/4$. Beam is considered rigid. Assume linear distribution of foundation contract pressure as for a rigid footing. Compute shear and moment in beam by simple statics.

TABLE 3 (continued)
Definitions and Procedures, Analysis of Beams on Elastic Foundation

Group 2 - Beams of medium length: $\pi/4 < \lambda\ell < \pi$. End conditions influence all sections of the beam. Compute moments and shears throughout the beam length by the infinite beam formulas, top panel of Figure 10. Determine in this way the shear and moments at the two ends of the beam. By superposing on the loaded beam two pairs of concentrated forces and moments at the ends of the beam, solutions for the infinite beam are modified to conform to the actual end conditions. For example, if Q = 0 and M = 0 at the ends of a free-ended beam, apply redundant shear and moment at the ends equal and opposite to that detemined from the infinite beam formulas. See reference cited in text for formulas for moments and shears in end loaded beam of finite length.

Group 3 - Long beams: $\lambda\ell > \pi$. End condition at distant end has negligible influence on moment and shear in the interior of the beam. Consider beam as extending an infinite distance away from loaded end. Compute moment and shear caused by interior loads by formulas for infinite beam, top panel of Figure 10. Compute moment and shear for loads applied near the beam ends by formulas for semi-infinite beam. bottom panel of Figure 10. Superpose moment and shear obtained from the two load systems.

4. Obtain functions $A_{\lambda x}$, $B_{\lambda x}$, $C_{\lambda x}$, $D_{\lambda x}$, for use in formulas of Figure 10 from Figure 11.

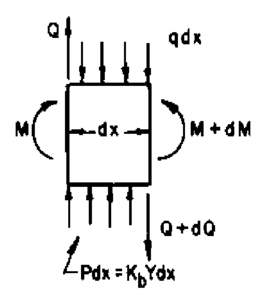
Sign Convention:

Consider infinitely small element of beam between two vertical cross sections at a distance d_x apart.

+Q = Upward acting shear force to left of section.

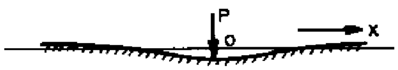
+M = Clockwise movement acting from the left to the section.

+y = Downward delection.



INFINITE BEAM

CONCENTRATED LOAD



DEFLECTION: $y = \frac{P\lambda}{2K} A_{\lambda x}$

MOMENT: $M = \frac{P}{4\lambda} C_{\lambda x}$

SHEAR: $Q = -\frac{P}{2} D_{\lambda x}$

APPLIED MOMENT

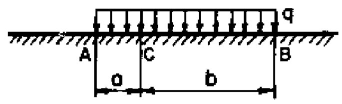
DEFLECTION: $y = \frac{M_o \lambda^2}{K} B_{\lambda x}$

MOMENT: $M = \frac{M_o}{2} D_{\lambda x}$

SHEAR: $Q = -\frac{M_o \lambda}{2} A_{\lambda x}$

UNIFORMLY DISTRIBUTED LOAD

POINT C IS UNDER LOAD

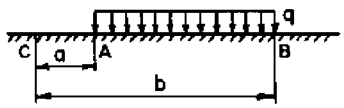


DEFLECTION: $y_C = \frac{q}{2K}(2 - D_{\lambda a} - D_{\lambda b})$

MOMENT: $M_C = \frac{q}{4\lambda^2}(B_{\lambda a} + B_{\lambda b})$

SHEAR: $Q_C = \frac{q}{4\lambda}(C_{\lambda a} - C_{\lambda b})$

POINT C IS LEFT OF LOAD

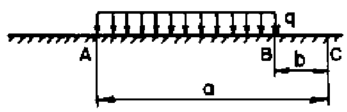


DEFLECTION: $y_C = \frac{q}{2K}(D_{\lambda a} - D_{\lambda b})$

MOMENT: $M_C = -\frac{q}{4\lambda^2}(B_{\lambda a} - B_{\lambda b})$

SHEAR: $Q_C = \frac{q}{4\lambda}(C_{\lambda a} - C_{\lambda b})$

POINT C IS RIGHT OF LOAD



DEFLECTION: $y_C = -\frac{q}{2K}(D_{\lambda a} - D_{\lambda b})$

MOMENT: $M_C = \frac{q}{4\lambda^2}(B_{\lambda a} - B_{\lambda b})$

SHEAR: $Q_C = \frac{q}{4\lambda}(C_{\lambda a} - C_{\lambda b})$

SEMI-INFINITE BEAM

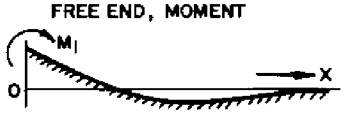
FREE END, CONCENTRATED LOAD

DEFLECTION: $y = \frac{2P_1\lambda}{K} D_{\lambda x}$

MOMENT: $M = -\frac{P_1}{\lambda} B_{\lambda x}$

SHEAR: $Q = -P_1 C_{\lambda x}$

FREE END, MOMENT

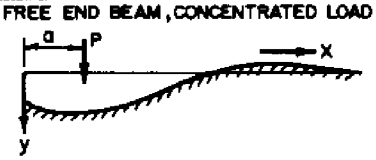


DEFLECTION: $y = -\frac{2M_1\lambda^2}{K} C_{\lambda x}$

MOMENT: $M = M_1 A_{\lambda x}$

SHEAR: $Q = -2M_1 \lambda B_{\lambda x}$

FREE END BEAM, CONCENTRATED LOAD NEAR END



DEFLECTION: $y = \frac{P\lambda}{2K}\left[(C_{\lambda a} + 2D_{\lambda a})A_{\lambda x} - 2(C_{\lambda a} + D_{\lambda a})B_{\lambda x} + A_{\lambda(a+x)}\right]$

IF NOTATION $(C_{\lambda a} + 2D_{\lambda a}) = \alpha$
AND $(C_{\lambda a} + D_{\lambda a}) = \beta$ IS USED

MOMENT: $M = \frac{P}{4\lambda}\left\{\alpha C_{\lambda x} - 2\beta D_{\lambda x} + C_{\lambda(a-x)}\right\}$

SHEAR: $Q = -\frac{P}{2}\left\{\alpha D_{\lambda x} - \beta A_{\lambda x} \pm D_{\lambda(a-x)}\right\}$

FIGURE 10
Computation of Shear, Moment, and Deflection, Beams on Elastic Foundation

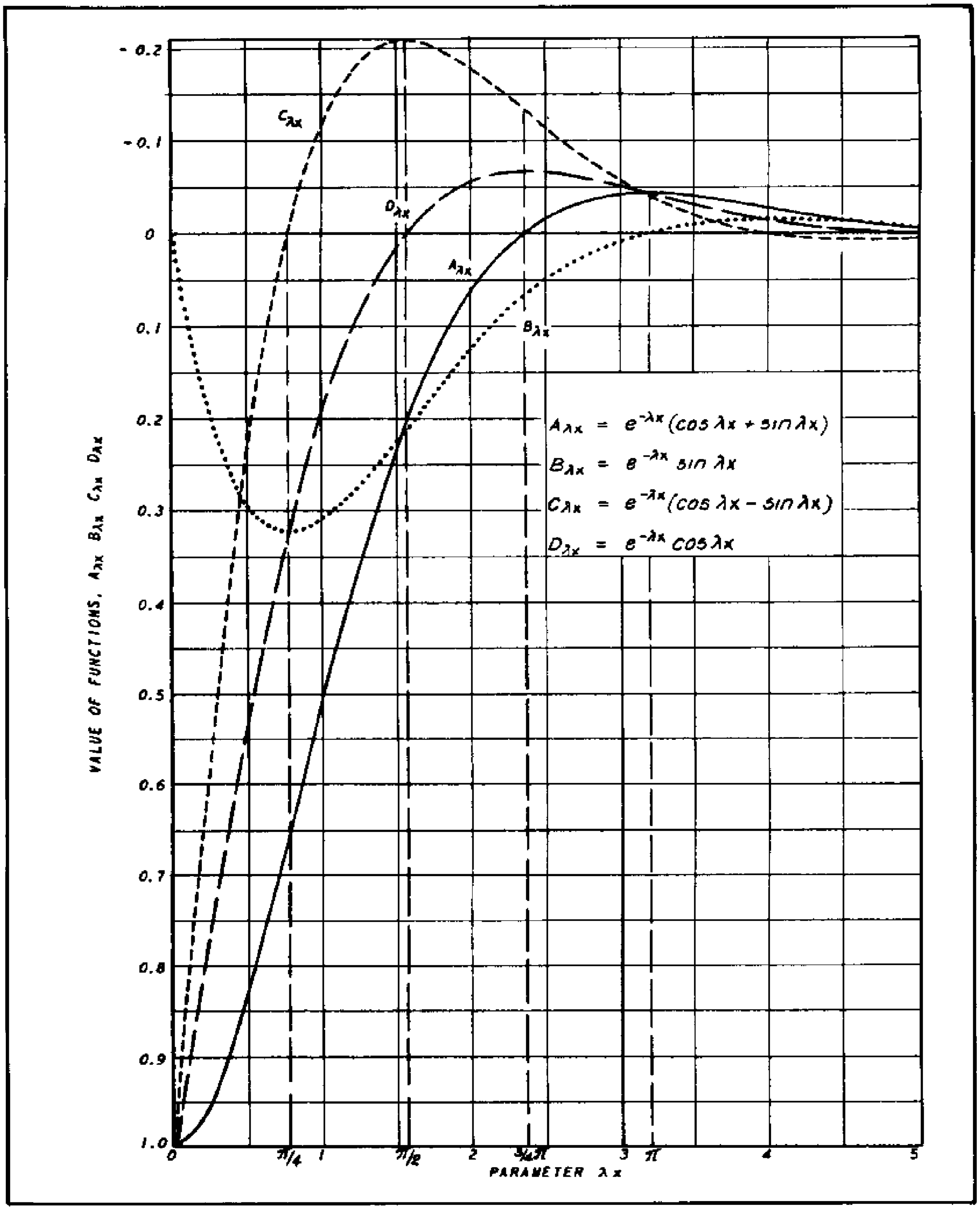


FIGURE 11
Functions for Shear, Moment, and Deflection, Beams on Elastic Foundations

TABLE 4

Definitions and Procedures, Mats on Elastic Foundations

Definitions:

r = Distance of point under investigation from point column load along radius

M_r, M_t = Radial and tangential moments (polar coordinates) for a unit width of mat

Q = Shear per unit width of mat

M_x = Moment which causes a stress in the x-direction (rectangular coordinates)

M_y = Moment which causes a stress in the y-direction (rectangular coordinates)

σ_x = Stress due to M_x

σ_y = Stress due to M_y

y = Deflection of mat at a point

b = width of mat

Procedure for Analysis:

1. Determine modulus of subgrade reaction for foundation width "b" - as follows:

   For cohesive soils: $K_b = K_{v1}/b$,

   For granular soils: $K_b = K_{v1}\left(\frac{b+1}{2b}\right)^2$

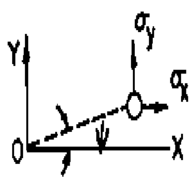
2. Determine mat thickness h from shear requirements at critical sections.
3. Determine values of E and Poisson's ratio μ for mat.
4. Calculate flexural rigidity of mat, $D = \frac{Eh^3}{12(1-\mu^2)}$
5. Calculate radius of effective stiffness: $L = \sqrt[4]{\frac{D}{K_b}}$
6. Radius of influence of individual column load equals approximately 4L.

7. To determine radial and tangential moments and deflections at any point from an interior column load use the following formulas:

$$M_r = -\frac{P}{4}\left[Z_4(\xi) - (1-\mu)\frac{Z_3'(\xi)}{\xi}\right],\quad \xi = \frac{r}{L},\quad Q = -\frac{P}{4L}\cdot Z_4'(\xi)$$

$$M_t = -\frac{P}{4}\left[\mu Z_4(\xi) + (1-\mu)\frac{Z_3'(\xi)}{\xi}\right],\quad y = \frac{PL^2}{4D} Z_3(\xi)$$

To convert radial and tangential moments to rectangular coordinates, use the following relationships:



$$M_x = M_r \cos^2\psi + M_t \sin^2\psi$$

$$M_y = M_r \sin^2\psi + M_t \cos^2\psi$$

Determine functions $Z_3(\xi)$, $Z'_3(\xi)$, $Z_4(\xi)$, and $Z'_4(\xi)$ from Figure 12.

[retrieve TABLE 4: (continued) Definitions and Procedures, Mats on Elastic Foundations]

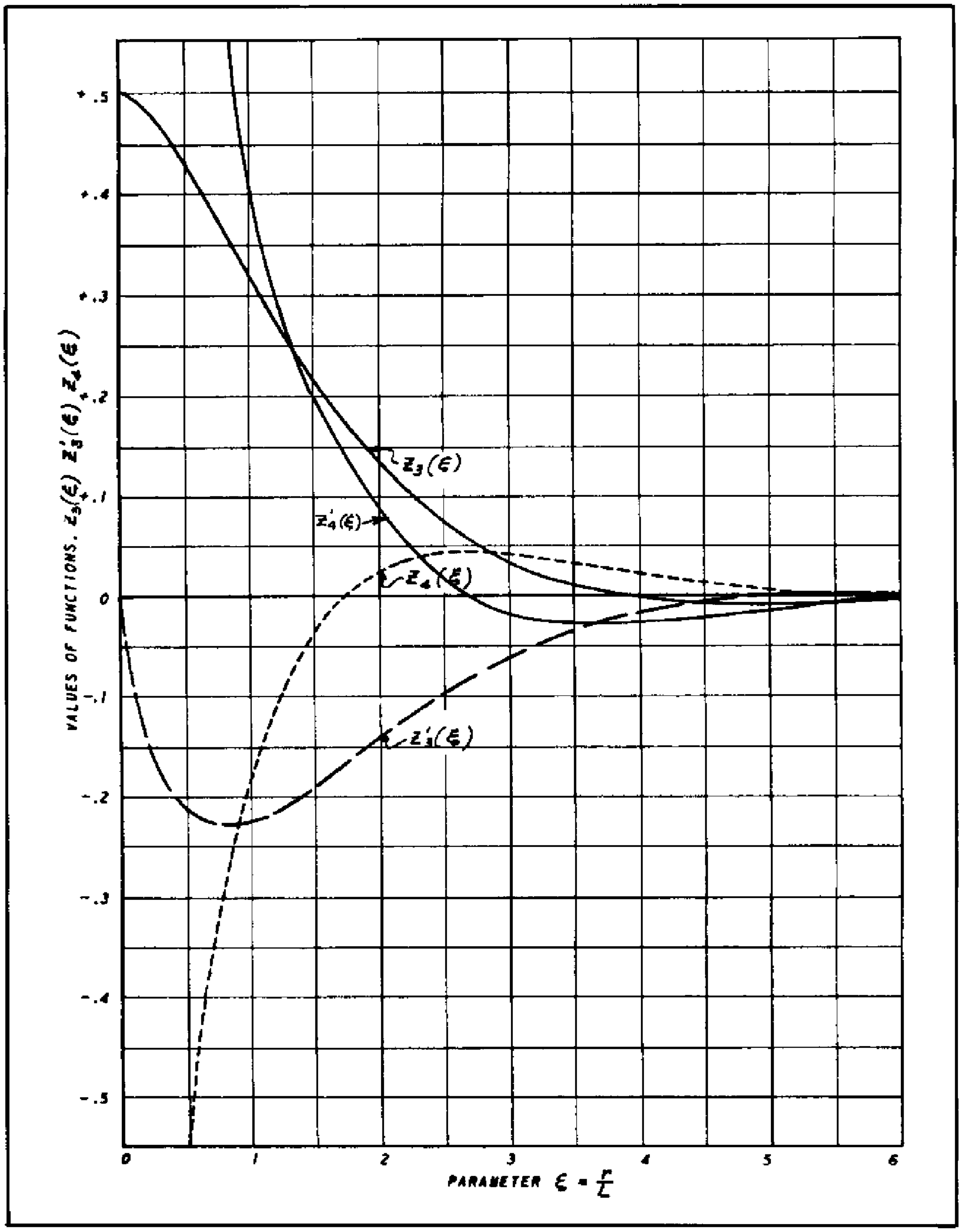


FIGURE 12
Functions for Shear, Moment, and Deflection, Mats on Elastic Foundation

c. <u>Modulus of Subgrade Reaction</u>. The modulus of subgrade reaction (K) is expressed as:

$$K = p/\Delta H$$

where:

p = contact pressure (stress unit)
Δ = soil deformation (length)

(1) K varies with the width and shape of the loaded area. Empirical correction for strip footings from Reference 12, <u>Evaluation of Coefficient of Subgrade Reaction</u>, by Terzaghi are:

(a) Cohesive soil.

$$K_b = K_{VI} / b$$

where: K_b = coefficient of subgrade reaction for foundation of width b

K_{VI} = coefficient of subgrade reaction for a 1' x 1' plate

If the loaded area is of width, b, and length, m_b, k_b assumes the value:

$$K_b = \frac{k_{VI}}{b} \left(\frac{m + 0.5}{1.5m} \right)$$

If actual plate load tests on cohesive soil are not available, estimates of K_v can be made in general accordance with the recommendations in Reference 12. If actual plate load tests are not available use correlation for K_{v*}, in Figure 6, DM-7.1, Chapter 5.

(b) Granular soil.

$$K_b = K_{VI} \left(\frac{b + 1}{2b} \right)^2$$

(c) Limitations. Values of K_b as determined from extrapolation of plate bearing tests should be utilized with judgement and care. Unlike the deformation in full size mat the deformation from plate load tests is not reflective of the underlying deeper strata. Also results from plate load tests on saturated or partially saturated clays may be unreliable because time may not permit complete consolidation of loaded clay.

(2) An estimate of K_b may be obtained by back calculating from a settlement analysis. The settlement of the mat can be calculated assuming a uniform contact pressure and utilizing the methods outlined in DM-7.1, Chapter 4 5. The contact pressure is then divided by the average settlement to obtain an estimate of K_b:

$$K_b = \frac{P}{\Delta H_{avg}}$$

where ΔH_{avg} = average computed settlement of the mat.

For a flexible circular mat resting on a perfectly elastic material [W-DELTA]H+avg, 0.85 x settlement at the center. For other shapes see DM-7.1, Chapter 5, Table 1.

d. Numerical Methods. Methods of analyses of mat foundation which account for the stiffness of the superstructure and the foundation, in which the soil is modelled as an elastic half space continuum utilizing finite element techniques are more accurate. A variety of soil constitutive relationships such as linear elastic, non-linear elastic, elasto-plastic, etc. can be utilized. Finite element techniques are well suited to these problems. See Appendix for listing of computer programs.

Section 5. FOUNDATIONS ON ENGINEERED FILL

1. UTILIZATION. Fills placed with controlled compaction may be used beneath structures for the following purposes:

(a) To raise the general grade of the structure or to replace unsuitable foundation soils.

(b) To provide a relatively stiff mat over soft subsoils in order to spread bearing pressures from column loads and decrease column settlements.

(c) To bridge over subsoils with erratic hard and soft spots or small cavities.

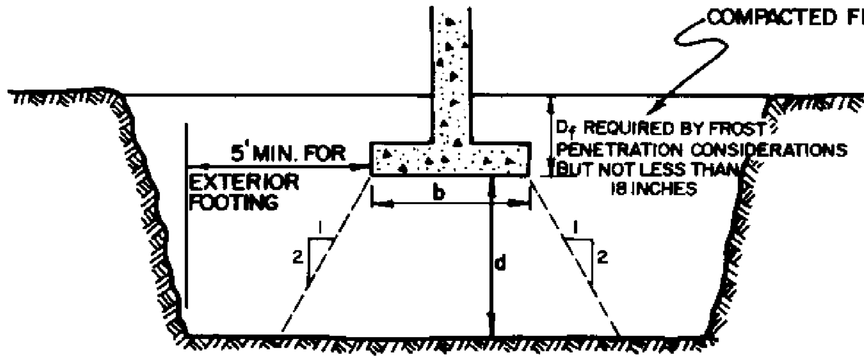
(d) To accelerate subsoil consolidation and to eliminate all or part of settlement of the completed structure when used with surcharge.

2. COMPACTION CONTROL. Rigidity, strength, and homogeneity of many natural soils may be increased by controlled compaction with appropriate equipment. A complete discussion of compaction requirements and control is presented in Chapter 2. Other methods of densifying in-place soils are given in DM-7.3, Chapter 2.

3. GEOMETRIC LIMITS OF COMPACTION. The limits of the zone of compacted soil beneath a footing should consider the vertical stresses imposed by the footing (stress-bulb) on the soils beneath it. Recommended requirements for compaction beneath a square and a continuous footing are illustrated in Figure 13. For large footings, the necessary depth of compacted fill should be determined from a settlement analysis.

Section 6. FOUNDATIONS ON EXPANSIVE SOILS

1. POTENTIAL EXPANSION CONDITIONS. Soils which undergo volume changes upon wetting and drying are termed expansive or swelling soils. If surface clays above the water table have a PI greater than about 22 (CH clays) and relatively low natural water content, potential expansion must be considered. These soils are most commonly found in arid climates with a deficiency of rainfall, over-evaporation, and where the groundwater table is low. Mottled, fractured,



CONTINUOUS FOOTING

d = DEPTH TO ADEQUATE BEARING MATERIAL } WHICHEVER IS LESS
d = 2 X b

SQUARE FOOTING

d = DEPTH TO ADEQUATE BEARING MATERIAL } WHICHEVER IS LESS
d = 1-1/2 X b

FIGURE 13
Limits of Compaction Beneath Square and Continuous Footings

or slickensided clays, showing evidence of past desiccation, are particularly troublesome. For other causes of swelling in soils and for the computations of resulting heave see DM-7.1, Chapter 5, and DM-7.3, Chapter 3 for further guidance.

2. ELIMINATING SOIL EXPANSION POTENTIAL. Where economically feasible, remove potentially expansive soils from beneath footings and replace with compacted fill of granular soils or nonexpansive materials. If this cannot be done, consider spread footings or drilled and underreamed caissons founded below the zone of active swelling. Design the shafts of such foundations with sufficient reinforcing to resist tensile forces applied to shaft by friction or adhesion in the swelling materials. Reinforcing must be carried into the belled section to a point 4" above the base. At any depth, tensile forces exerted on a shaft equal circumferential area of the shaft times the difference between average swelling pressure above and below the point under consideration.

Placing the base of foundation near the water table reduces heave damage because of little change in moisture content. For construction techniques in such soil see Figure 14 (top and center, Reference 13, Soil Mechanics and Foundation, by Parcher and Means), DM-7.3, Chapter 3, and Reference 14, Design and Performance of Mat Foundation on Expansive Clay, by Lytton and Woodburn.

Footing foundations can be successful if sufficient dead load is exerted to eliminate heave completely or reduce it significantly in conjunction with a structure rigid enough to withstand stress due to heaving. See DM-7.1 Chapter 5, and DM-7.3, Chapter 3 for methods of estimating the magnitude of swell.

3. MINIMIZING EXPANSION EFFECTS. Where it is not economically leasible to remove expansive materials or to support foundations below depths of possible expansion, the effects can be minimized as follows:

(a) Where large seasonal changes in soil moisture are responsible for swelling, schedule construction during or immediately after a prolonged rainy period when there will be less potential volume change in the future.

(b) For concrete floor slabs placed directly on potentially expansive clays, provide expansion joints so the floor can move freely from the structural frame.

(c) For foundations on fill materials containing plastic fines and susceptible to swelling, place fill at moisture content above optimum with density no higher than required for strength and rigidity. Excessive compaction will result in greater swelling.

(d) Grade beams should contain sufficient steel reinforcement to resist the horizontal and vertical thrust of swelling soils. If practical, place compressible joint filler or open blocks or boxes beneath grade beams to minimize swelling pressures.

(e) Provide impervious blankets and surface grading around the foundations to prevent infiltration of surface water.

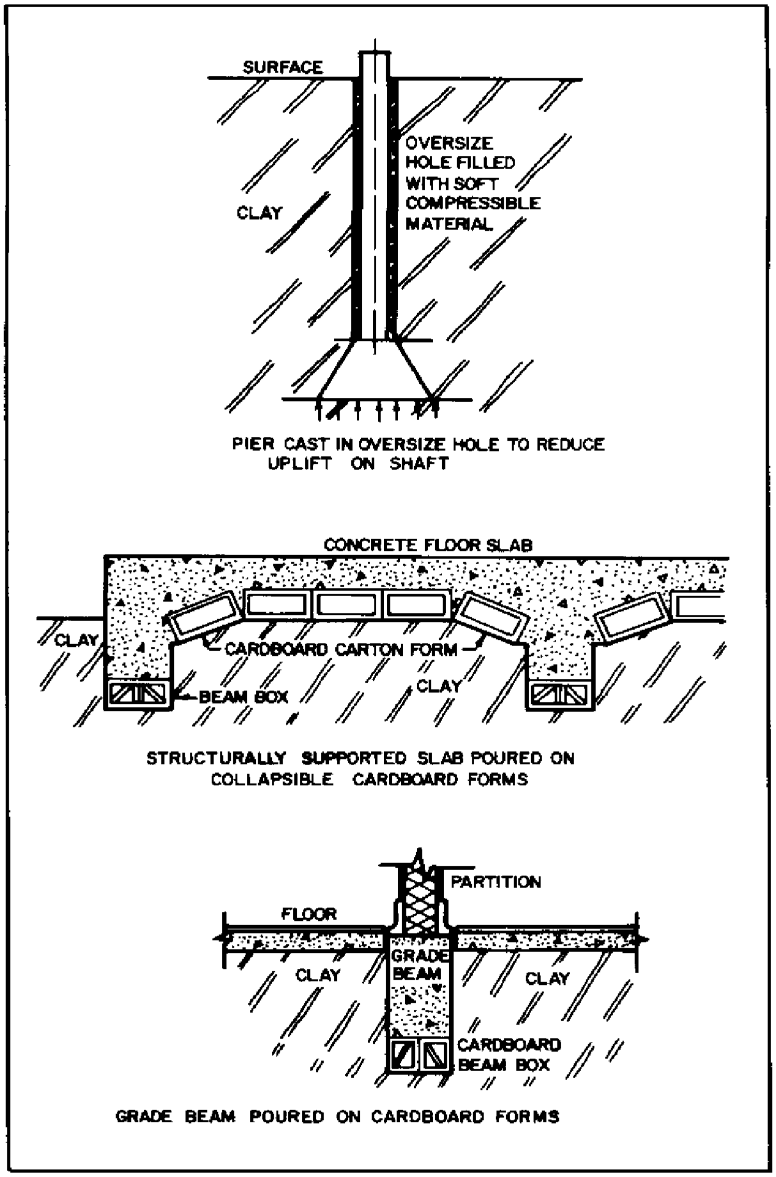


FIGURE 14
Construction Details for Swelling Soils

(f) Locate water and drainage lines so that if any leakage occurs, water will not be readily accessible to foundation soils thereby causing damage.

(g) Consider stabilization of the foundation soils and backfill mate materials by lime and other agents.

For further guidance see Reference 15, <u>Foundations on Expansive Soils</u>, by Chen, and DM-7.3, Chapter 3.

4. COLLAPSING SOILS. Many collapsing soils will slake upon immersion, but this is not a definitive indicator. Definite identification requires a pair of consolidation tests with and without saturation, or by plate load tests where water is added with the plate under stress. In the case of collapsible soil, the e-log p curve for the specimen, which was allowed to come in contact with water, is below that of the dry specimen. See DM-7.1, Chapter 3 for testing procedures.

(a) If positive measures are practical for avoiding water foundation contact, the "dry" strength of soil can be used for design purposes.

(b) Alternately, under some conditions, prewetting of the soil is found effective in reducing settlements. By this process, the soil structure breaks down resulting in its densification. This increases its strength and reduces the total and differential settlement. This method is not very successful especially where little additional load is applied during wetting. For further guidance see DM-7.3, Chapter 3, and Reference 7.

Section 7. FOUNDATION WATERPROOFING

1. APPLICATIONS. See Table 5 for general requirements for waterproofing, dampproofing, and waterstops. See References 16, 17, and 18; <u>Foundation Design</u>, by Teng, NAVFAC TS-07110, <u>Membrane Waterproofing</u>, and NAVFAC TS-07160, <u>Bituminous Dampproofing</u>, respectively, for guidance. For basements below ground, two general schemes are employed as follows:

(a) Where the permanent water table is above the top of basement slab, provide pressure resistant slab (pressure slab) or relieve uplift pressures by underdrainage (relieved slab).

(b) Where the water table is deep but infiltration of surface water dampens backfill surrounding basement, provide dampproof walls and slabs (see Table 5, Dampproofing).

2. PRESSURE AND RELIEVED SLABS.

a. <u>Pressure Slabs</u>. In general, the choice between pressure or relieved slab depends on overall economy, maintenance, layout, and operation, and must be evaluated individually for each project. For basements extending only a small depth below groundwater, a pressure slab to resist maximum probable hydrostatic uplift usually is economical. Also, when the soil below water level is very pervious, an extensive and consequently very costly drainage system may be necessary. See Case A, Figure 15. Drainage material should be

TABLE 5

Requirements for Foundation Waterproofing and Dampproofing

Type	Materials	Workmanship	Applicability	Remarks
Waterproofing				
1. Membrane	Bitumen: 1) ASTM D449, Type [A] [B] [C] Asphalt, ASTM D450, Type II Coal-tar; 2) Bituminous plastic cement; Federal Specifications SS-C-153, Type I for asphalt, Type II for coal-tar; 3) Felt or fabric material impregnated with asphalt or coal-tar as specified in references cited in text; 4) for primer, protective covering, prefabricated laminated asphalt waterproofing see references.	Before starting the work inspect all surfaces to be waterproofed to determine that they are in satisfactory condition. Complete conduit, piping, and other required rough-in. Start after all defects and unsatisfactory conditions have been corrected. Surfaces to be treated should be clean and dry, smooth and free from deleterious and excess materials and projections. Use priming coat of creosote and asphalt at no less than one gallon per 100 sq ft on surface receiving coal-tar membrane waterproofing and asphalt membrane water proofing respectively. For membrane application, use at least 3-ply for dampproofing and 5-ply for hydrostatic pressure. Apply membrane using shingle method. For detailed requirements see references.	Use on exterior wall surfaces, over roofs or underground structures, for patching openings through walls formed for utilities or structural members. Method is frequently utilized, but careful inspection and control is required to obtain completely satisfactory application.	Vulnerable to damage. Hard to locate and repair damaged area.

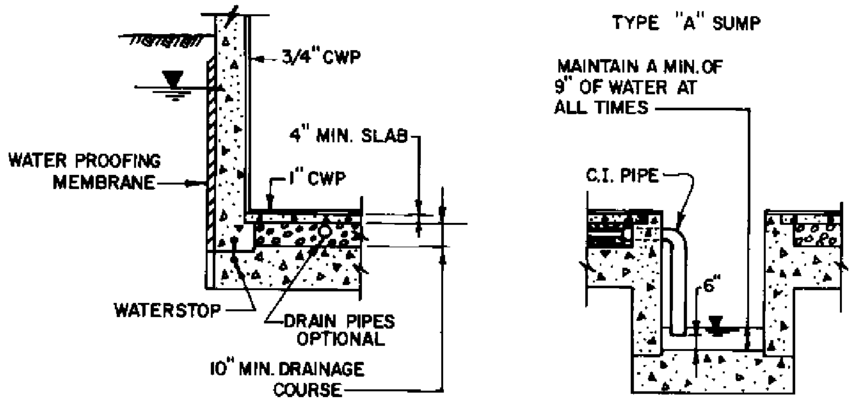
TABLE 5 (continued)
Requirements for Foundation Waterproofing and Dampproofing

Type	Materials	Workmanship	Applicability	Remarks
2. Cement plaster	One part Portland cement, no more than two parts of sand and no more than two parts of water. Sand should contain no sizes smaller than No. 200 sieve and preferably is well graded between No. 100 and No. 8 sieve sizes. Waterproofing compounds are optional, except that no salts or deliquescent materials are permitted.	All surfaces in contact with form shall be entirely chipped away. Floor concrete shall have rake finish. All faces shall be rinsed thoroughly with clean water. Wall and ceiling coast shall be applied in 2 coats that together total betwen 5/8 to 3/4 inch in thickness. Floors to have one coat of 1 inch thickness. All surfaces are to be floated with wood float and hand finished by steel troweling.	Used on exposed interior surfaces of walls, floors and occasionally on ceilings where the ceiling is exposed on the outside to water-pressures. Appropriate for highest type of basement occupancy. Care is required to obtain a seal surrounding wall openings for utilities, etc.	Can resist high hydrostatic pressures without injury. Easily inspected for imperfections and can be easily repaired.
Dampproofing				
1. Interior faces	Coating consisting of finely divided iron mixed with sand, cement, and oxidizing agent.	Surfaces to be thoroughly cleaned and roughened. Apply in at least four brush coats.	Used on basement walls below ground at damp or wet locations, below temporary groundwater levels, or under hydrostatic heads of only several feet.	Lower cost. If appearance of interior surfaces is important, use cement plaster waterproofing.

TABLE 5 (continued)
Requirements for Foundation Waterproofing and Dampproofing

Type	Materials	Workmanship	Applicability	Remarks
2. Exterior faces	Hot coal tar, straight run, pitch, Type B coating, or asphalt Type B mopping. Built-up in successive coats to a minimum of 1/8 inch thickness.	Concrete and masonry surfaces to be dry and free from dust, dirt, grease, oil, or other coatings before application. Use primary cost of creosote and asphalt at no less than one gallon per 100 sq ft as surface receiving coal-tar pitch dampproofing and asphalt or fibrous asphalt dampproofing, respectively. Either the hot application method using asphalt or coal tar bitumen or the coal application method using fibrous asphalt may be used. For further details on application method and protective covering see references.	Used on basement walls below ground at damp or wet locations, below temporary groundwater levels, or under hydrostatic heads of only several feet.	Lower cost. If appearance of interior surfaces is important, use cement plaster water-proofing.

Ⓐ PRESSURE SLAB

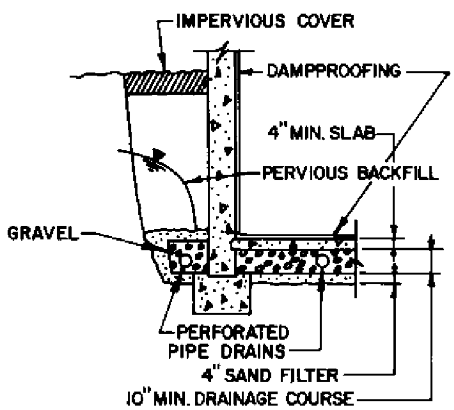


GENERAL REQUIREMENTS:

1. SEE CHAPTER 6 DM-7.1 FOR REQUIREMENTS FOR FILTER IN DRAINAGE SYSTEM.
2. SEE TABLE 5 FOR REQUIREMENTS FOR CEMENT PLASTER WATERPROOFING (CWP), AND DAMPPROOFING.

MATERIAL FOR UNDER-FLOOR DRAINAGE COURSE SHALL CONSIST OF SOUND, CLEAN GRAVEL OR CRUSHED ROCK, 3/4 IN. TO 2 IN. IN SIZE.

Ⓑ RELIEVED SLAB



USE TYPE "A" OR "C" SUMP

FOR PRESSURE RELIEVED SLAB, PROVIDE PERIPHERAL DRAIN AT BASE OF FOUNDATION WALL. REPLACE CWP ON FOUNDATION WALL WITH DAMPPROOFING.

FIGURE 15
Typical Foundation Drainage and Waterproofing

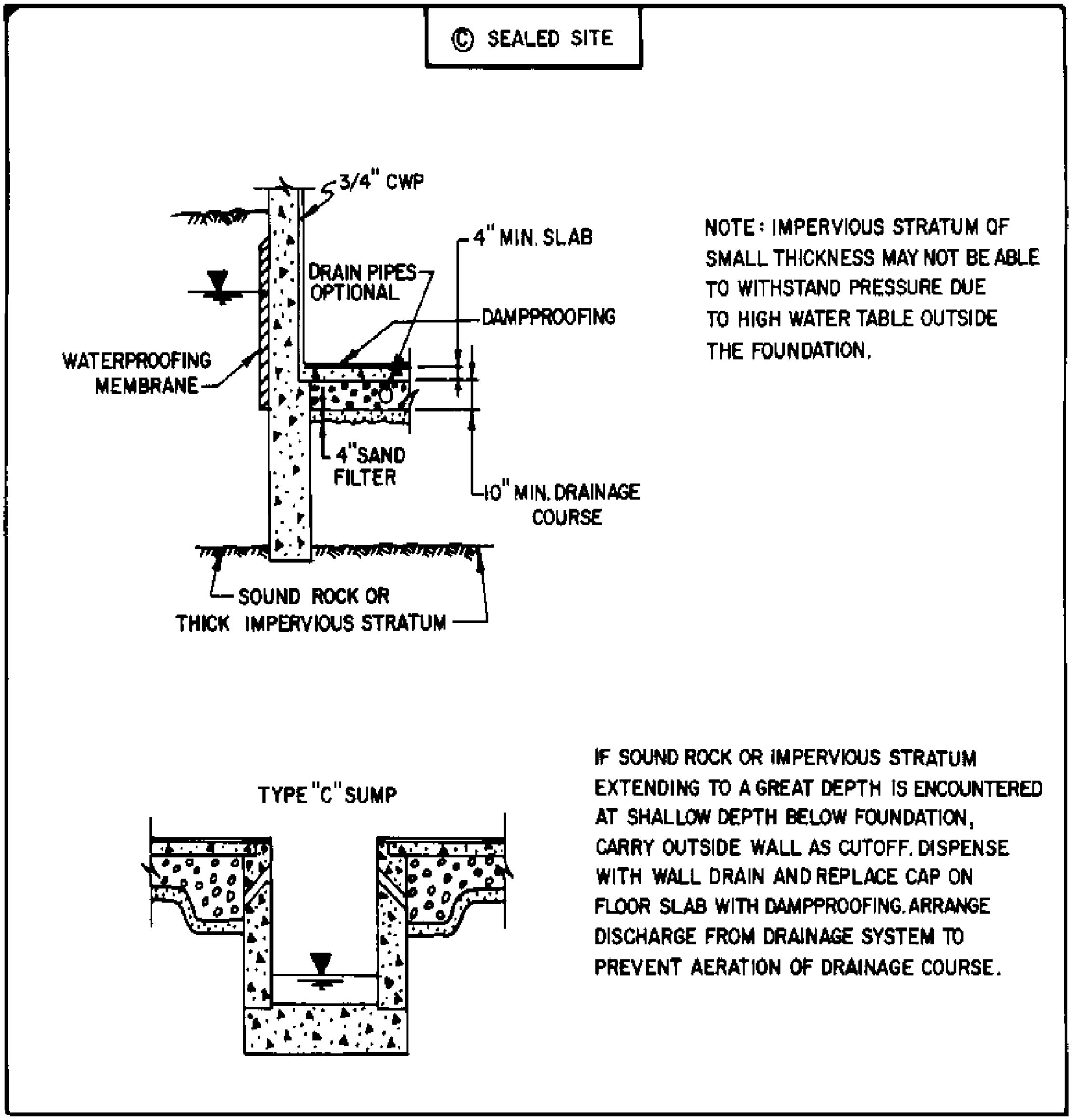


FIGURE 15 (continued)
Typical Foundation Drainage and Waterproofing

sound, clean gravel or crushed stone graded between 3/4 and 2 inches, compacted by two or three coverages of vibrating base plate compactor. Open joint drain pipe should be added beneath slabs of large plan dimensions. Provide water- stops at the construction joints between pressure slab and wall.

b. Relieved Slabs. For basements at considerable depth below groundwater level, it is usually economical to provide pressure relief beneath the foundation slab. See Cases B and C, Figure 15. If pervious materials of great depth underlie the foundation level, include a wall drain and drainage course beneath the slab. See DM-7.1, Chapter 6 for filter requirements and drain spacing. If foundation walls can be carried economically to underlying sound impervious rock or thick impervious stratum, omit wall drains. Arrange sumps for drainage discharge to avoid aerating drainage course.

3. WATERPROOFING REQUIREMENTS. In addition to leakage under pressure through joints and cracks, water may move through basement walls and floors by capillary action and as water vapor. A drainage course can be used to interrupt capillary action, but it will not prevent movement of water vapor through slabs. Plastic vapor barriers are useful in providing an effective vapor barrier.

a. Membrane Waterproofing and Dampproofing. Apply membrane (see Figure 15B) for basements utilized for routine purposes where appearances are unimportant and some dampness is tolerable.

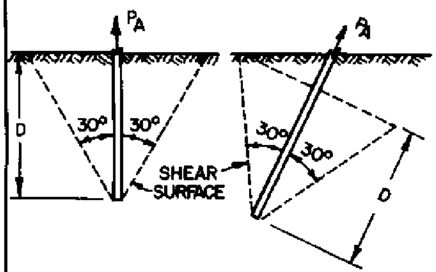
b. Cement Plaster Waterproofing. Where it is important to prevent dampness or moisture in a basement, specify cement plaster waterproofing, consisting of sand-cement mortar hand troweled on chipped and roughened concrete surface. Properly applied, this is a very effective method against dampness and moisture.

Section 8. UPLIFT RESISTANCE

1. ROCK FOUNDATION. Resistance to direct uplift of tower legs, guys, and antennas, where the foundation is resting directly over rock, may be provided by reinforcing bars grouted in rock. In the absence of pullout tests, determine uplift resistance by empirical formulas of Figures 16 and 18. These formulas apply to bars in fractured rock near the rock surface. Higher shear strength is to be expected in sound, unweathered rock. To develop rock strength, sufficient bond must be provided by grout surrounding the bar. Bond strengths May be increase by using washers, rock bolts, deformed bars, or splayed bar ends.

Guidance for design rules is given in DM-7.3, Chapter 3 and quality control associated with pre-stressed, cement grouted rock anchors is found in Reference 19, Rock Anchors - State of the Art, by Littlejohn and Bruce.

2. SOIL FOUNDATION. For sustained uplift on a footing, see Table 2. Transient uplift from live loads applied to footings, piers, posts or anchors is



SINGLE BAR ANCHORAGES

P_A = ALLOWABLE ANCHOR PULL

D = EMBEDMENT DEPTH, MEASURED AS SHOWN

C_{all} = ALLOWABLE ROCK SHEAR STRESS

f_s = ALLOWABLE BAR STRESS, 20 KSI

b_{rqd} = BOND STRESS ON BAR PERIMETER REQUIRED TO DEVELOP C_{all}

A = BAR CROSS-SECTION AREA

$P_A = (2.1)\, D^2 (C_{all})$ AND $P_A = A\, f_s$

$$b_{rqd} = \frac{P_A}{\text{BAR PERIMETER} \times D}$$

TESTS INDICATE THAT FOR BAR IN ORDINARY FRACTURED ROCK NEAR THE SURFACE:

MINIMUM D(FT) = $(1.25)\sqrt{P_A}$ (KIPS)

AT THIS DEPTH C_{all} = 0.3 KSF AND SHOULD NOT BE TAKEN GREATER THAN THIS VALUE WITHOUT PULLOUT TESTS

SPACING OF BARS IN PLAN SHOULD EXCEED 1.2D

EXAMPLE:

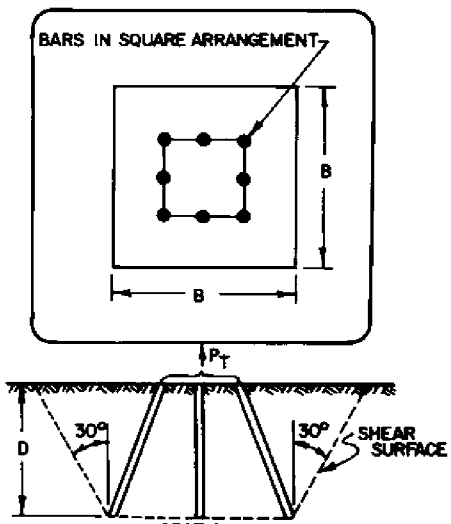
GIVEN: P_A = 20K FOR 1 IN. SQUARE BAR

MINIMUM $D = 1.25\sqrt{20} = 5.6$ FT.

BAR SPACING = 1.2 (5.6) = 6.7 FT.

$$b_{rqd} = \frac{20{,}000}{4(5.6)(12)} = 74 \text{ PSI}$$

PLAN



SECTION

BAR GROUP ANCHORAGE

P_T = ALLOWABLE ANCHOR PULL FOR GROUP OF BARS.

N = NUMBER OF BARS IN SQUARE ARRANGEMENT

$P_T = 4.6D\,(B + 0.58D)\, C_{all}$ AND

$P_T = NA\, f_s$

$$b_{rqd} = \frac{P_T}{\text{BAR PERIMETER} \times ND}$$

TESTS INDICATE THAT FOR BAR GROUP IN ORDINARY FRACTURED ROCK NEAR THE SURFACE:

MINIMUM D(FT)

$$D = \frac{-4.6\,B\,C_{all} + \sqrt{21.2\,B^2 (C_{all})^2 + 10.7\,C_{all} \times NAf_s}}{5.34\,C_{all}}$$

AT THIS DEPTH C_{all} = 0.3 KSF AND SHOULD NOT BE TAKEN GREATER THAN THIS VALUE WITHOUT PULLOUT TESTS.

EXAMPLE:

GIVEN P_T = 80K, USE 4 – 1 IN SQUARE BARS

B = 4.5 FT f_s = 20 KSI

MIN. D: WITHOUT TESTS:

$$D = \frac{-4.6 \times 4.5 \times 0.3 + \sqrt{21.2 \times 4.5^2 \times 0.3^2 + 10.7 \times 0.3 \times 4 \times 1 \times 200}}{5.34 \times 0.3}$$

$= 6.9$ FT

$$b_{rqd} = \frac{80{,}000}{(4)(4)(6.9)(12)} = 60 \text{ PSI}$$

FIGURE 16
Capacity of Anchor Rods in Fractured Rock

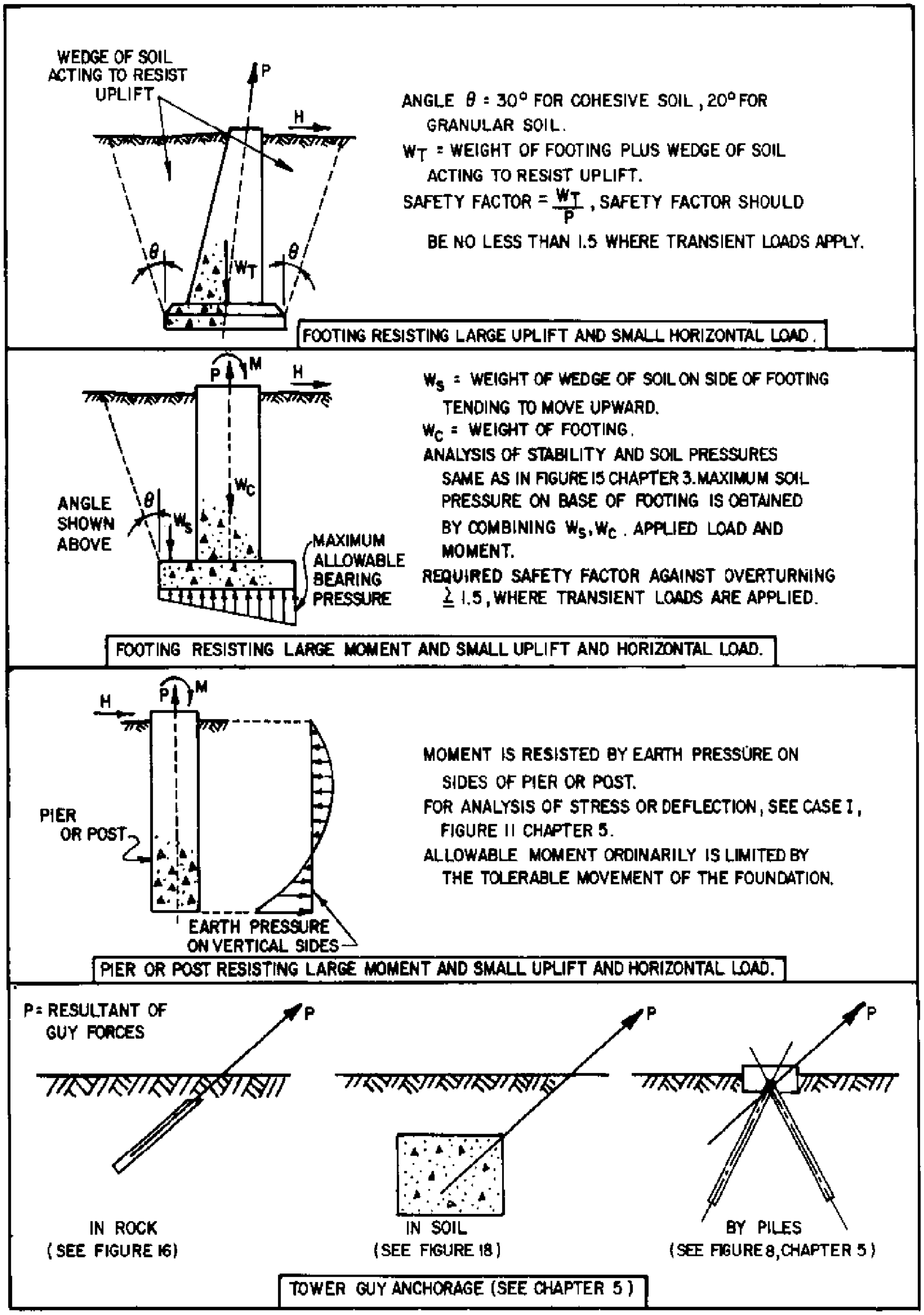


FIGURE 17
Resistance of Footings and Anchorages to Combined Transient Loads

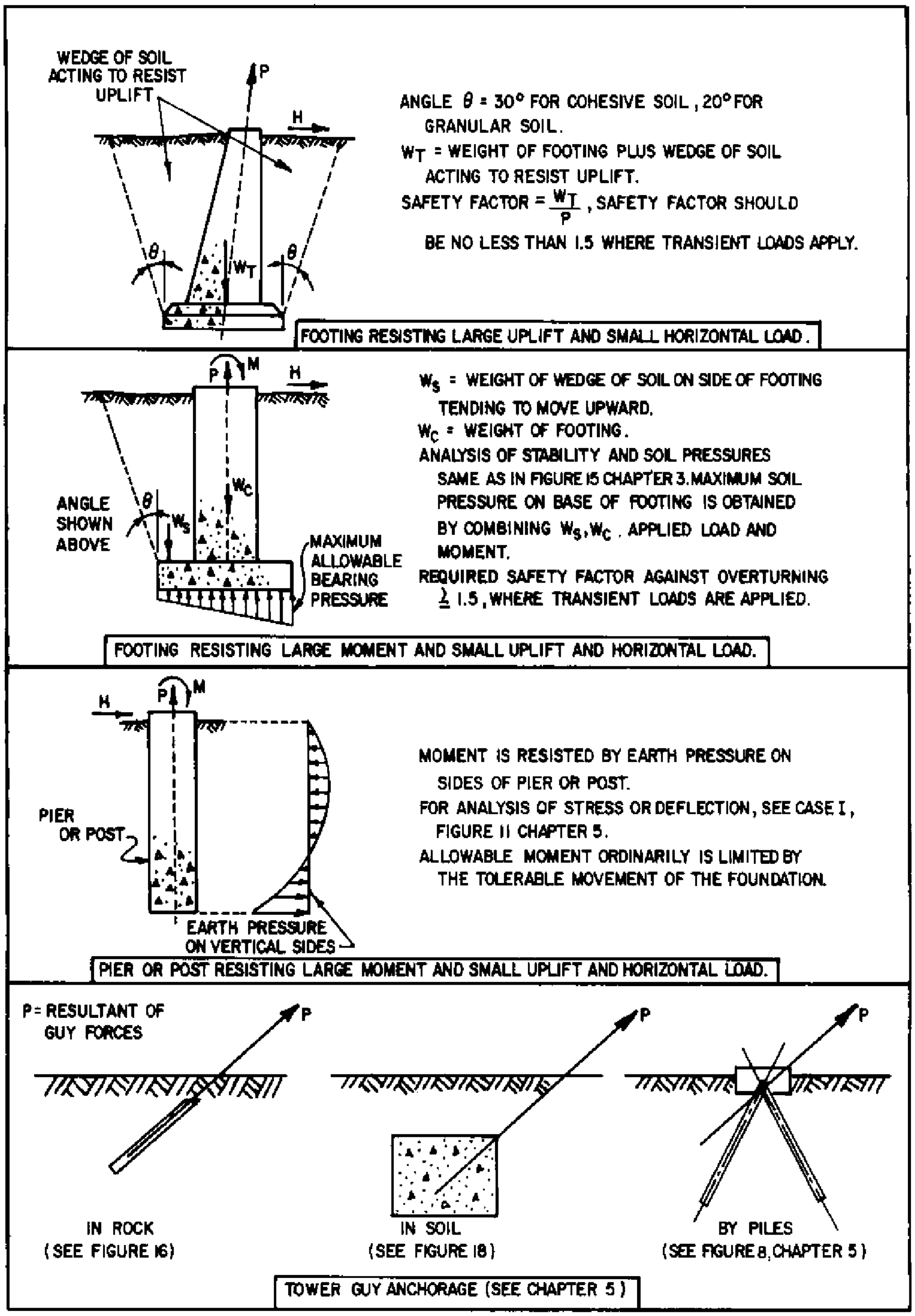


FIGURE 17
Resistance of Footings and Anchorages to Combined Transient Loads

analyzed as shown in Figure 17. Tower guy anchorage in soil is analyzed in Figure 18. For a deadman in weak soil, it may be feasible to replace a considerable volume of soil with granular backfill and construct the block within the new backfill. If this is done, the passive wedge should be contained entirely within the granular fill, and the stresses on the remaining weak material should be investigated. See Reference 6 for guidance.

3. CORROSION. For temporary anchors minimal protection is needed unless the environments are such that rapid deterioration takes place. Permanent anchor bars are covered with grout. In corrosive environments it is common practice to provide additional protection by coating with material (epoxy, polyester resin) with proven resistance to existing or anticipated corrosive agents. The coating agent should not have any adverse effect on the bond.

4. ROCK AND SOIL ANCHORS. When the load to be resisted is large, wire tendons which can also be prestressed to reduce movements are employed.

Also, because of corrosion special precautions may be necessary when permanent anchors are provided in marine environments. In the analysis of anchors, because of submergence, the bouyant unit weight of soils should be used. The buildup of excess pore pressure due to repetitive loads should also be evaluated in the case of granular soils. For a discussion of cyclic mobility and liquefaction see DM-7.3, Chapter 1. For the design of anchors see DM-7.3, Chapter 3.

REFERENCES

1. Meyerhof, G.G., Influence of Roughness of Base and Ground Water Condition on the Ultimate Bearing Capacity of Foundations, Geotechnique, 1955.

2. Meyerhof, G.G., The Bearing Capacity of Foundations Under Eccentric and Inclined Loads, Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Zurich, 1953.

3. Meyerhof, G.G., The Ultimate Bearing Capacity of Foundations on Slopes, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

4. Button, S.F., The Bearing Capacity of Footings on a Two-Layer Cohesive Subsoil, Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Zurich, 1953.

5. Vesic, A. S. Bearing Capacity of Shallow Foundations, Foundation Engineering Handbook, Winterkorn and Fang, eds., Van Nostrand Reinhold Company, New York, Chapter 3, 1975.

6. Bowles, J.E., Foundation Analysis and Design, McGraw Hill Book Co., New York, pp. 137-139, 1977

7. Peck, R.B., Hanson, W.E. and Thornburn, T.H., Foundation Engineering, 2nd Edition, John Wiley & Sons, Inc., New York, 1974.

8. Canadian Geotechnical Society, Shallow Foundations, Canadian Foundation Engineering Manual, Montreal, Canada, Part 2, 1978.

9. CP2004, Sulphates in Soils and Groundwaters, Classification and Recommendations, BRS Digest, No. 90, Second Series, London, England, 1972.

10. Hetenyi, M., Beams on Elastic Foundation, University of Michigan Press, Ann Arbor, MI, 1946.

11. ACI Committee 436, Suggested Design Procedures for Combined Footings and Mats, American Concrete Institute, 1966.

12. Terzaghi, K., Evaluation of Coefficient of Subgrade Reaction, Geotechnique, Vol. 5, No. 4, 1955.

13. Parcher, J.V. and Means, R.E., Soil Mechanics and Foundations, Charles E. Merril Publishing Company, Columbus, OH, 1968.

14. Lytton, R.L. and Woodburn, J.A., Design and Performance of Mat Foundations on Expansive Clay, Proceedings of the Third International Conference on Expansive Soils, 1973.

15. Chen, F.H., Foundations on Expansive Soils, Footing Foundations, Elsevier Scientific Publishing Co., New York, Chapter 5, 1975.

16. Teng, W.C., Foundation Drainage and Waterproofing, Foundation Design, Prentice Hall, Englewood Cliffs, NJ, Chapter 5, 1962.

17. NAVFAC TS-07110, Membrane Waterproofing, Naval Facilities Engineering Command, 1979.

18. NAVFAC TS-07160, Bituminous Dampproofing, Naval Facilities Engineering Command, 1978.

19. Littlejohn, G.S. and Bruce, D.A., Rock Anchors-State of the Art, Foundation Publications Ltd., England, 1977.

20. NAVFAC DM-2.02, Loads, Naval Facilities Engineering Command, 1986.

Copies of Guide Specifications and Design Manuals may be obtained from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120.

CHAPTER 5. DEEP FOUNDATIONS

Section 1. INTRODUCTION

1. SCOPE. This chapter presents information on the common types of deep foundations, analysis and design procedures, and installation procedures. Deep foundations, as used in this chapter, refer to foundations which obtain support at some depth below the structure, generally with a foundation depth to width ratio (D/B) exceeding five. These include driven piles, drilled piles, drilled piers/caissons, and foundations installed in open or braced excavations well below the general structure. Diaphragm walls are discussed in DM-7.03, Chapter 3.

2. APPLICATION. Deep foundations are used in a variety of applications including:

(a) To transmit loads through an upper weak and/or compressible stratum to underlying competent zone.

(b) To provide support in areas where shallow foundations are impractical, such as underwater, in close proximity to existing structures, and other conditions.

(c) To provide uplift resistance and/or lateral load capacity.

3. RELATED CRITERIA. For additional criteria relating to the design of deep foundations and the selection of driving equipment and apparatus, see the following sources:

Subject	Source
Pile Driving Equipment	NAVFAC DM-38.04
General Criteria for Piling in Waterfront Construction	NAVFAC DM-25.06

4. LOCAL PRACTICE. The choice of the type of deep foundation such as pile type(s), pile design capacity, and installation procedures is highly dependent on local experience and practice. A design engineer unfamiliar with these local practices should contact local building/engineering departments, local foundation contractors, and/or local foundation consultants.

5. INVESTIGATION PROGRAM. Adequate subsurface exploration must precede the design of pile foundations. Investigations must include the following:

(a) Geological section showing pattern of major strata and presence of Possible obstructions, such as boulders, buried debris, etc.

(b) Sufficient test data to estimate Strength and compressibility parameters of major strata.

(c) Determination of probable pile bearing stratum.

For field explorations and testing requirements, see DM-7.01, Chapter 2.

6. CONSTRUCTION INSPECTION. The performance of a deep foundation is highly dependent on the installation procedures, quality of workmanship, and installation/design changes made in the field. Thus, inspection of the deep foundation installation by a geotechnical engineer normally should be required.

Section 2. FOUNDATION TYPES AND DESIGN CRITERIA

1. COMMON TYPES. Tables 1 and 2 summarize the types of deep foundations, fabricated from wood, steel, or concrete, in common usage in the United States. Table 1 presents pile types and Table 2 presents excavated foundation types including drilled piers/caissons. General comments on applicability of the various foundation types are given in Table 2, but local experience and practices, comparative costs, and construction constraints should be reviewed carefully for each site.

a. Driven Piles. These are piles which are driven into the ground and include both low displacement and high displacement piles. Low displacement piles include H and I section steel piles. Open end piles which do not form a plug, jetted piles, and pre-bored driven piles may function as low displacement piles. Solid section piles hollow section closed end piles, and open end piles forming a soil plug function as high displacement piles. All the pile types in Table 1 except auger-placed piles are driven piles.

b. Excavated Foundations. These foundations include both drilled piles and piers and foundations constructed in open or braced excavations (see Reference 1, Foundation Design, by Teng). Drilled piles include auger-placed piles and drilled pier/caissons either straight shaft or belled.

2. OTHER DEEP FOUNDATION TYPES. Tables 1 and 2 include only the most commonly used pile types and deep foundation construction procedures. New and innovative types are being developed constantly, and each must be appraised on its own merits.

a. Drilled-in Tubular Piles. These consist of heavy-gauge steel tubular pile capable of being rotated into the ground for structure support. Soils in the tube may be removed and replaced with concrete. Used in penetration of soil containing boulders and obstructions, or drilling of rock socket to resist uplift and lateral forces. Steel H-sections within concrete cores are used to develop full end bearing for high load capacity.

b. TPT (Tapered Pile Tip) Piles. These consist of a mandrel drive corrugated shell with an enlarged precast concrete base. This type of pile is usually considered in conditions suitable for pressure injected footings. The principal claimed advantage is the avoidance of punching through a relatively thin bearing stratum.

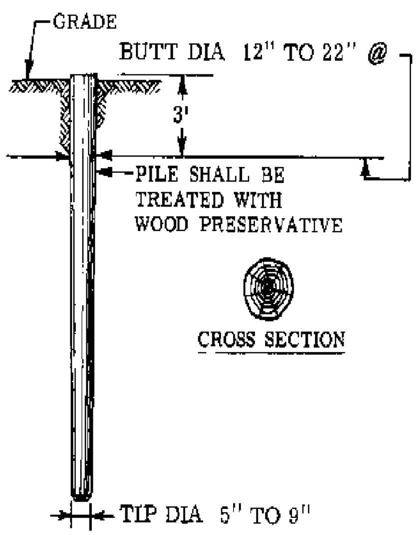
c. Interpiles. These consist of an uncased concrete pile, formed by a mandrel riven steel plate. A steel pipe mandrel of smaller diameter than the plate is used, and the void created by the driven plate is kept continuously filled with concrete. It is claimed that this pile develops greater side friction in a granular soil than drilled piers and conventional driven piles.

TABLE 1
Design Criteria for Bearing Piles

PILE TYPE	TIMBER	STEEL - H SECTIONS
CONSIDER FOR LENGTH OF	30-60FT	40-100 FT
APPLICABLE MATERIAL SPEC-IFICATIONS.	ASTM -D25	ASTM-A36
MAXIMUM STRESSES.	MEASURED AT MOST CRITICAL POINT, 1200 PSI FOR SOUTHERN PINE AND DOUGLAS FIR. SEE U.S. D.A. WOOD HANDBOOK NO.72 FOR STRESS VALUES OF OTHER SPECIES.	12,000 PAI.
CONSIDER FOR DESIGN LOADS OF	10 - 50 MNS	40 -120 TONS
DISADVANTAGES	DIFFICULT TO SPLICE. VULNERABLE TO DAMAGE IN HARD DRIVING, TIP MAY HAVE TO BE PROTECTED. VULNERABLE TO DECAY UNLESS TREATED, WHEN PILES ARE INTERMITTENTLY SUBMERGED.	VULNERABLE TO CORROSION WHERE EXPOSED HP SECTION MAY BE DAMAGED OR DEFLECTED BY MAJOR OBSTRUCTIONS.
ADVANTAGES	COMPARATIVELY LOW INITIAL COST. PERMANENTLY SUBMERGED PILES ARE RESISTANT TO DECAY, EASY TO HANDLE.	EASY TO SPLICE. AVAILABLE IN VARIOUS LENGTHS AND SIZES. HIGH CAPACITY. SMALL DISPLACEMENT.

		ABLE TO PENETRATE THROUGH LIGHT OBSTRUCTIONS. HARDER OBSTRUCTIONS MAY BE PENETRATED WITH APPROPRIATE POINT PROTECTION OR WHERE PENETRATION OF SOFT ROCK IS REQUIRED.
REMARKS	BEST SUITED FOR FRICTION PILE IN GRANULAR MATERIAL.	BEST SUITED FOR ENDBEARING ON ROCK. REDUCE ALLOWABLE CAPACITY FOR CORROSIVE LOCATIONS.

TYPICAL ILLUSTRATIONS.



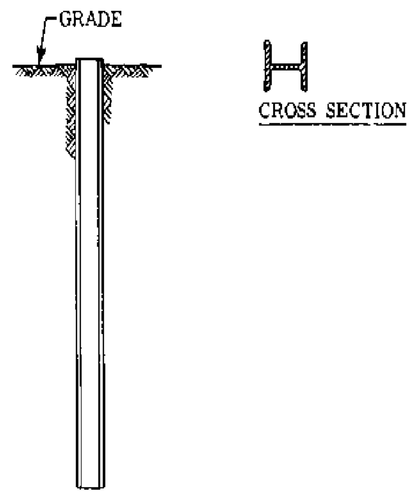


TABLE 1 (continued)
Design Criteria for Bearing Piles

PILE TYPE	PRECAST CONCRETE (INCLUDING PRESTRESSED)	CAST-IN-PLACE CONCRETE (THIN SHELL DRIVEN WITH MANDREL)
CONSIDER FOR LENGTH OF	40-50 FT FOR PRECAST 60-100 FT FOR PRESTRESSED.	10-120 FT BUT TYPICALLY IN THE 50-90 FT.RANGE
APPLICABLE MATERAL SPEC-IFICATIONS,	ACI 318 FOR CONCRETE ASTM A15-FOR REINFORCING STEEL	ACI CODE 318-FOR CONCRETE.
MAXIMUM STRESSES.	FOR PRECAST-33 % OF 28 DAY STRENGTH OF CONCRETE. FOR PRESTRESSED- F+c,=0.33 F'+c,-0.27F+pe, (WHERE: F+pe, IS THE EFFECTIVE PRESTRESS STRESS ON THE GROSS SECTION). SPECIFICALLY DESIGNED FOR A WIDE RANGE OF LOADS.	33% OF 28-DAY STRENGTH OF CONCRETE, WITH INCREASE TO 40% OF 28 DAY STRENGTH. PROVIDING: (A) CASING IS A MINIMUM 14 GAUGE THICKNESS (B) CASING IS SEAMLESS OR WITH WELDED SEAMS (C) RATIO OF STEEL YIELD STRENGTH TO CONCRETE 28 DAY STRENGTH IS NOT LESS THAN 6. (D) PILE DIAMETER IS NOT GREATER THAN 17". SPECIFICALLY DESIGNED FOR A WIDE RANGE OF LOADS.
DISADVANTAGES	UNLESS PRESTRESSED, VULNERABLE TO HANDLING RELATIVELY HIGH BREAKAGE RATE ESPECIALLY WHEN PILES ARE TO TO BE SPLICED. HIGH INITIAL COST. CONSIDERABLE DISPLACEMENT PRESTRESSED DIFFICULT TO SPLICE.	DIFFICULT TO SPLICE AFTER CONCRETING. REDRIVING NOT RECOMMENDED. THIN SHELL VULNERABLE DURING DRIVING TO EXCESSIVE EARTH PRESSURE OR IMPACT. CONSIDERABLE DISPLACEMENT.
ADVANTAGES	HIGH LOAD CAPACITIES. CORROSION RESISTANCE CAN BE ATTAINED. HARD DRIVING POSSIBLE.	INITIAL ECONOMY. TAPERED SECTIONS PROVIDE HIGHER BEARING RESISTANCE IN GRANULAR STRATUM CAN BE INTERNALLY INSPECTED AFTER DRIVING RELATIVELY LESS WASTE STEEL MATERIAL. CAN BE DESIGNED AS END BEARING OR FRICTION PILE, GENERALLY WADED IN THE 40-100 TON RANGE.
REMARKS	CYLINDER PILES IN PARTICULAR ARE SUITED FOR BENDING RESISTANCE. GENERAL LOADING RANGE IS 40-400 TONS.	BEST SUITED FOR MEDIUM LOAD FRICTION PILES IN GRANULAR MATERIALS.

TYPICAL ILLUSTRATIONS

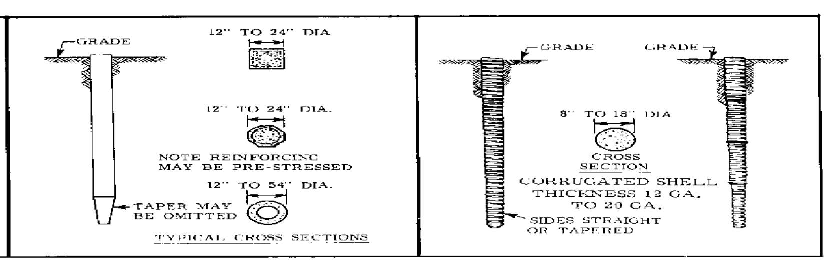
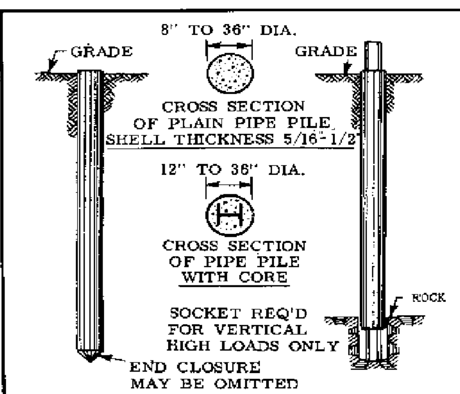


TABLE 1 (continued)
Design Criteria for Bearing Piles

PILE TYPE	CAST-IN-PLACE CONCRETE PILES (SHELLS DRIVEN WITHOUT MANDREL)	PRESSURE INJECTED FOOTINGS
CONSIDER FOR LENGTH OF	30-80 FT.	10 TO 60 FT
APPLICABLE MATERIAL SPECIFI-CATION	ACI CODE 318	ACI CODE 318
MAXIMUM STRESSES	33% OF 28 - DAY STRENGTH OF CONCRETE 9,000 PSI IN SHELL, THAN 1/8 INCH THICK.	33% OF 28-DAY STRENGTH OF CONCRETE. 9,000 PSI FOR PIPE SHELL IF THICKNESS GREATER THAN 1/8 INCH
CONSIDER FOR DESIGN LOADS OF	50-70 TONS	60-120 TONS.
DISADVANT-AGES	HARD TO SPLICE AFTER CONCRETING. CONSIDERABLE DISPLACEMENT	BASE OF FOOTING CANNOT BE MADE IN CLAY OR WHEN HARD SPOTS E.G. ROCK LEDGES) ARE PRESENT IN SOIL PENETRATED. WHEN CLAY LAYERS MUST BE PENETRATED TO REACH SUITABLE MATERAL, SPECIAL PRECAUTIONS ARE REQUIRED FOR SHAFTS IF IN GROUPS.
ADVANTAGES	CAN BE REDRIVEN. SHELL NOT EASILY DAMAGED.	PROVIDES MEANS OF PLACING HIGH CAPACITY FOOTINGS ON BEARING STRATUM WITHOUT NECESSITY FOR EXCAVATION OR DEWATERING. HIGH BLOW ENERGY AVAILABLE FOR OVERCOMING OBSTRUCTIONS. GREAT UPLIFT RESISTANCE IF SUITABLY REINFORCED.
REMARKS	BEST SUITED FOR FRICTION PILES OF MEDIUM LENGTH.	BEST SUITED FOR GRANULAR SOILS WHERE BEARING IS ACHIEVED THROUGH COMPACTION AROUND BASE. MINIMUM SPACING 4'-6" ON CENTER.

TYPICAL ILLUSTRATIONS



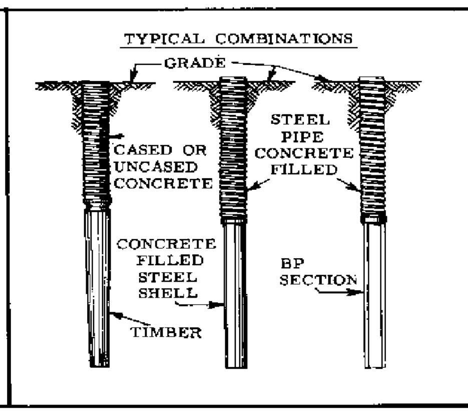


TABLE 1 (continued)
Design Criteria for Bearing Piles

PILE TYPE	CONCRETE FILLED STEEL PIPE PILES	COMPOSITE PILES
CONSIDER FOR LENGTH OF	40-120 FT OR MORE	60-200 FT
APPLICABLE MATERIAL SPECIFIC-ATIONS	ASTM A36-FOR CORE. ASTM A252-FOR PIPE. ACI CODE 318-FOR CONCRETE.	ACI CODE 318-FOR CONCRETE ASTM A36-FOR STRUCTURAL SECTION. ASTM A252-FOR STEEL PIPE. ASTM D25 -FOR TIMBER.
MAXIMUM STRESSES.	9,000 PSI FOR PIPE SHELL 33% OF 28-DAY STRENGTH OF CON-CRETE. 12,000 PSI ON STEEL CORES OF STRUCTURAL REINFORCING STEEL.	33% OF 28-DAY STRENGTH OF CONCRETE. 9,000 PSI FOR STRUCTURAL AND PIPE SECTIONS. SAME AS TIMBER PILES FOR WOOD COMPOSITE.
CONSIDER FOR DESIGN WAD OF	80-120 TONS WITHOUT CORES. 500-1,500 TONS WITH CORES.	30-100 TONS.
DISADVANT-AGES	HIGH INITIAL COST DISPLACEMENT FOR CLOSED END PIPE.	DIFFICULT TO ATTAIN GOOD JOINT BETWEEN TWO MATERIALS EXCEPT FOR PIPE COMPOSITE PILE.
ADVANTAGES	BEST CONTROL DURING INSTALLATION. NO DISPLACEMENT FOR OPEN END INSTALLATION. OPEN END PIPE BEST AGAINST OBSTRUCTIONS. CAN BE CLEANED OUT AND DRIVEN FURTHER. HIGH LOAD CAPACITIES. EASY TO SPLICE.	CONSIDERABLE LENGTH CAN BE PROVIDED AT COMPARATIVELY LOW COST. FOR WOOD COMPOSITE PILES. HIGH CAPACITY FOR PIPE AND HP COMPOSITE PILES. INTERNAL INSPECTION FOR PIPE COMPOSITE PILES.
REMARKS	PROVIDES HIGH BENDING RESISTANCE WHERE UNSUPPORTED LENGTH IS LOADED LATERALLY.	THE WEAKEST OF ANY MATERIAL USED SHALL GOVERN ALLOWABLE STRESSES AND CAPACITY.

TABLE 1 (continued)
Design Criteria for Bearing Piles

PILE TYPE	AUGER-PLACED, PRESSURE-INJECTED CONCRETE PILES	GENERAL NOTES
CONSIDER FOR LENGTH OF	30-60 FT.	1. STRESSES GIVEN FOR STEEL PILES ARE FOR NONCORROSIVE LOCATIONS, ESTIMATE POSSIBLE REDUCTION IN STEEL CROSS SECTION OR PROVIDE PROTECTION FROM CORROSION. 2. LENGTHS AND LOADS INDICATED ARE FOR FEASIBILITY GUIDANCE ONLY. THEY GENERALLY REPRESENT TYPICAL CURRENT PRACTICE, GREATER LENGTHS ARE OFTEN USED. 3. DESIGN LOAD CAPACITY SHOULD BE DETERMINED BY SOIL MECHANICS PRINCIPLES, LIMITING STRESSES IN PILES, AND TYPE AND FUNCTION OF STRUCTURE. SEE TEXT
APPLICABLE MATERIAL SPECIFIC-ATIONS	ACI-318	
MAXIMUM STRESSES.	3% OF 28-DAY STRENGTH OF CONCRETE	
CONSIDER FOR DESIGN LOAD OF	35-70 TONS	
DISADVANT-AGES	MORE THAN AVERAGE DEPENDENCE ON QUALITY WORKMANSHIP. NOT SUITABLE THRU PEAT OR SIMILAR HIGHLY COMPRESSIBLE MATERIAL. REQUIRES RELATIVELY MORE EXTEN-SIVE SUBSURFACE INVESTIGATION.	
ADVANTAGES	ECONOMY. COMPLETE NONDISPLACEMENT MINIMAL DRIVING VIBRATION TO ENDANGER ADJACENT STRUCTURES. HIGH SKIN FRICTION. GOOD CONTACT ON ROCK FOR END BEAR-ING. CONVENIENT FOR LOW-HEADROOM UNDERPINNING WORK. VISUAL INSPECTION OF AUGERED MATER-IAL. NO SPLICING REQUIRED.	
REMARKS	BEST SUITED AS A FRICTION PILE.	

TYPICAL ILLUSTRATIONS

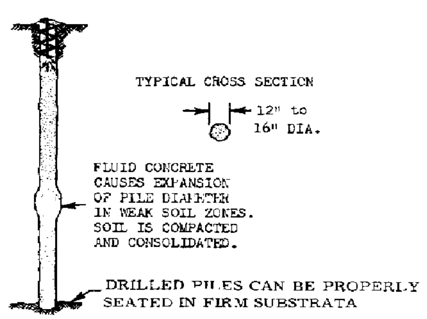


TABLE 2
Characteristics of Common Excavated/Drilled Foundations

1. PIERS (also called Shafts)

a. Description and Procedures - Formed by drilling or excavating a hole, removing the soil, and filling with concrete. Casing may be necessary for stabilization, and/or to allow for inspection and may or may not be pulled as the concrete is poured. Types include straight shaft piers and belled or underreamed piers. Drilled shaft diameters are typically 18 to 36 inches but can exceed 84 inches; belled diameters vary but are generally not larger than 3 times the diameter of the shaft. Excavated piers can be larger (shaft diameters exceeding 12 feet with belled diameters exceeding 30 feet have been constructed). Lengths can exceed 200 feet. Pier size depends on design load and allowable soil loads.

b. Advantages

- Completely non-displacement.
- Excavated material can be examined and bearing surface can be visually inspected in cased piers exceeding 30 inches in diameter (or smaller using TV cameras).
- Applicable for a wide variety of soil conditions.
- Pile caps usually not needed since most loads can be carried on a single pier.
- No driving vibration.
- With belling, large uplift capacities possible.
- Design pier depths and diameters readily modified based on field conditions.
- Can be drilled into bedrock to carry very high loads.

c. Disadvantages

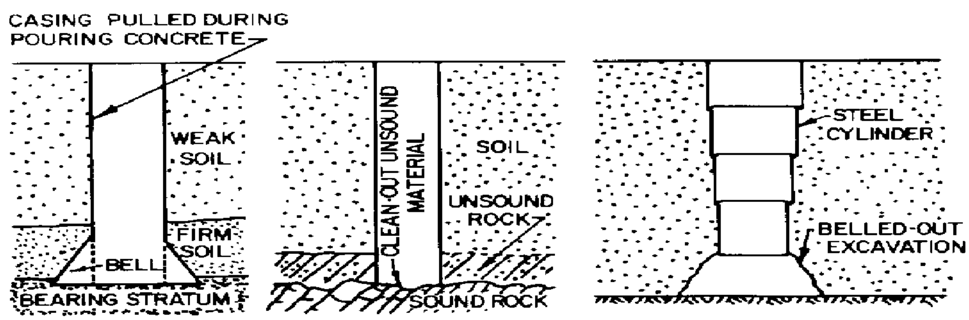
- More than average dependence on quality of workmanship; inspection required.
- Danger of lifting concrete when pulling casing can result in voids or inclusions of soil in concrete.
- Loose granular soils below the water table can cause construction problems.

TABLE 2 (continued)
Characteristics of Common Excavated/Drilled Foundations

- o Bell usually cannot be formed in granular soils below the water table.

- o Small diameter piers (less than 30 inches) cannot be easily inspected to confirm bearing and are particularly susceptible to necking problems.

d. Typical Illustration



2. INTERNALLY-BRACED COFFERDAM IN OPEN WATER

a. Description and Procedures - Generally only applicable if structure extends below mudline.

(1) Cofferdam constructed and dewatered before pouring of foundation.

(a) Install cofferdam and initial bracing below water in existing river/sea bottom. Cofferdam sheeting driven into bearing strata to control underseepage.

(b) Pump down water inside cofferdam.

(c) Excavate to bearing stratum completing bracing system during excavation.

(d) Construct foundation within completed and dewatered cofferdam.

(e) Guide piles or template required for driving cofferdams.

(f) Cofferdam designed for high water, ice forces, or load of floating debris.

(g) Cellular wall or double-wall cofferdams will eliminate or reduce required bracing system.

TABLE 2 (continued)
Characteristics of Common Excavated/Drilled Foundations

(2) Cofferdam excavated underwater

(a) Install cofferdam and initial bracing below water to existing river/sea bottom.

(b) Excavate underwater and place additional bracing to subgrade in bearing stratum.

(c) Seal bottom with tremie mat of sufficient weight to balance expected hydrostatic uplift.

(d) Pump out cofferdam and erect remainder of foundation structure.

(e), (f) and (g) same as dewatered cofferdam.

(h) Relief of water pressures below tremie slab may be used to decrease weight of tremie slab.

b. Advantages - Generally more economical than caissons if foundation is in less than 40 feet of water.

c. Disadvantages - Requires complete dewatering or tremie mat.

d. Typical Illustration

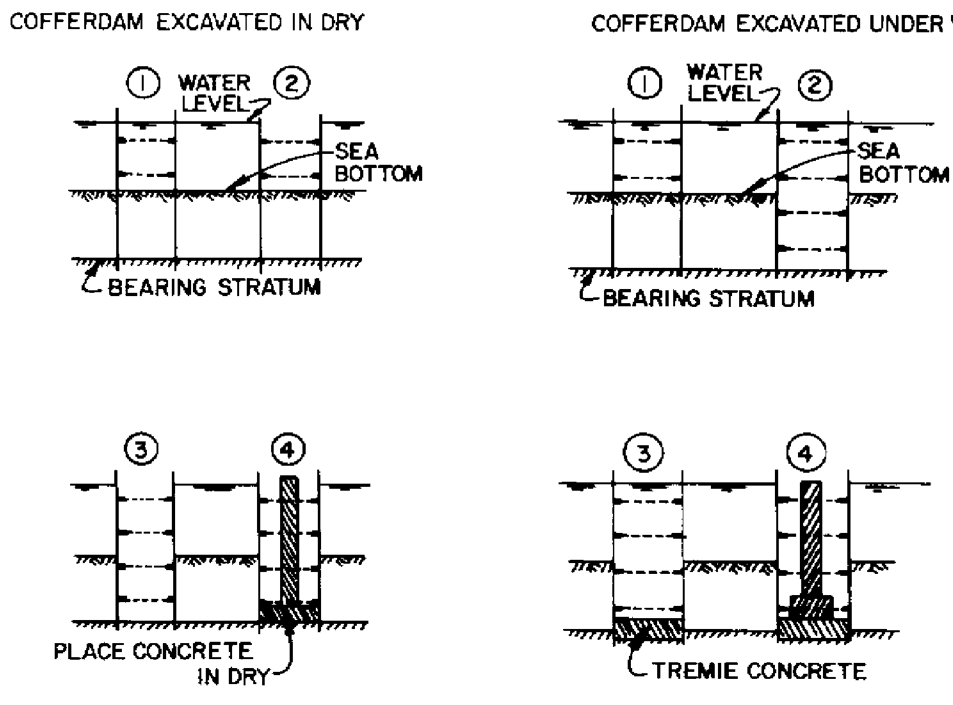


TABLE 2 (continued)
Characteristics of Common Excavated/Drilled Foundations

3. OPEN CAISSON

a. Description and Procedure - An open box or circular section with a cutting shoe on its lower edge. The caisson is sunk into place under its own weight by removal of the soil inside the caisson, jetting on the outside wall is often used to facilitate the process.

(1) Caissons should be considered when one or more of the following conditions exist:

(a) A substructure is required to extend to or below the river/sea bed.

(b) The soil contains large boulders which obstruct penetration of piles or drilled piers.

(c) The foundation is subject to very large lateral forces.

If these conditions do not exist the use of a caisson is not warranted because it is generally more expensive than other types of deep foundations. In open water, if the bearing stratum is less than about 40 feet below the water surface, a spread footing foundation constructed within cofferdams is generally less expensive.

(2) General method of construction includes:

(a) Float caisson shell into position.

(b) Build up shell in vertical lifts and place fill within shell until it settles to sea bottom.

(c) Continue buildup and excavate by dredging within caisson so as to sink it through unsuitable upper strata.

(d) Upon reaching final elevation in bearing stratum, pour tremie base.

(e) Provide anchorage or guides for caisson shell during sinking.

(f) Floating and sinking operations can be facilitated by the use of false bottoms or temporary domes.

(g) Dredging operations may be assisted by the use of jets or airlifts.

TABLE 2 (continued)
Characteristics of Common Excavated/Drilled Foundations

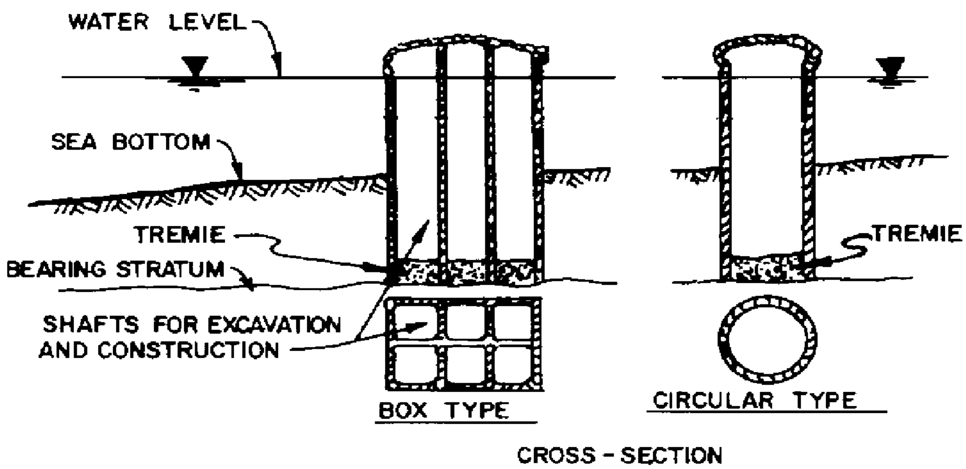
Generally appropriate for depths exceeding 50 to 60 feet and when final subgrade in the bearing stratum is not threatened by uplift from underlying pervious strata.

b. Advantages - Feasibility of extending to great depths.

c. Disadvantages

- o Bottom of the caisson cannot be thoroughly cleaned and inspected.
- o Concrete seal placed in water is not as satisfactory as placed in the dry.
- o Soil directly under the haunched portion near the cutting edges may require hand excavation by diver.
- o Construction is slowed down if obstruction of boulders or logs is encountered.

d. Typical Illustration



4. PNEUMATIC CAISSON

a. Description and Procedure - Similar to an open caisson but the box is closed and compressed air is used to keep water and mud from flowing into the box. Because of high costs, it is generally only used on large projects where an acceptable bearing stratum cannot be reached by open caisson methods because of excessive depth of water.

(1) Generally required for sinking to great depths where inflow of material during excavation can be damaging to surrounding areas and/or where uplift is a threat from underlying pervious strata.

TABLE 2 (continued)
Characteristics of Common Excavated/Drilled Foundations

(2) General method of construction includes:

(a) Float caisson into position.

(b) Build, up on top of caisson in vertical lifts until the structure settles to sea bottom.

(c) Continue buildup and excavate beneath the caisson, using compressed air when passing through unstable strata.

(d) Pour concrete base in the dry upon reaching final position in the bearing stratum.

(e) Provide anchorage or guides for caisson during sinking. For excavation in the dry, air pressure is generally made equal to total head of water above bottom of caisson.

b. Advantages

- o All work is done in the dry; therefore, controls over the foundation preparation and materials are better.
- o Plumbness of the caisson is easier to control as compared with the open caisson.
- o Obstruction from boulders or logs can be readily removed. Excavation by blasting may be done if necessary.

c. Disadvantages

- o The construction cost is high due to the use of compressed air.
- o The depth of penetration below water is limited to about 120 feet (50 psi). Higher pressures are beyond the endurance of the human body.
- o Use of compressed air restricts allowable working hours per man and requires strict safety precautions.

d. Typical Illustration

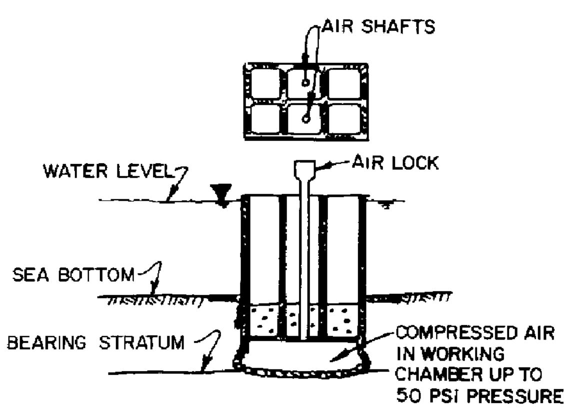


TABLE 2 (continued)
Characteristics of Common Excavated/Drilled Foundations

5. CAISSON (Floating Caisson)

a. Description and Procedure - Essentially a cast-on-land floating foundation sunk into position by backfilling.

(1) Used primarily for wharfs, piers, bulkheads, and breakwaters in water not more than 40 feet deep.

(2) General construction method includes:

(a) Prepare subgrade at sea bottom by dredging, filling, or combination of dredging and filling.

(b) Float caisson into position.

(c) Sink caisson to prepared foundation at the sea bottom by use of ballast.

(d) Provide anchorage or guides to protect floating caisson against water currents.

(e) Backfill for suitable foundation should be clean granular material and may require compaction in place under water.

b. Advantages

o The construction cost is relatively low.

o Benefit from precasting construction.

o No dewatering necessary.

c. Disadvantages

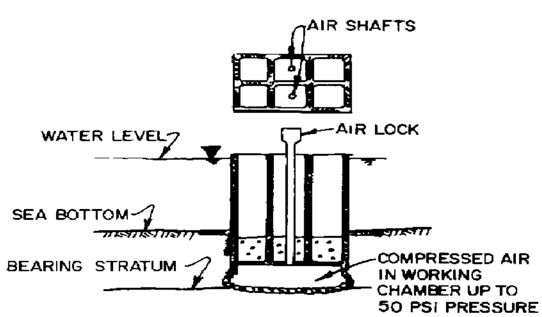
o The ground must be level or excavated to a level surface.

o Use is limited to only those conditions where bearing stratum is close to ground surface.

o Provisions must be made to protect against undermining by scour.

o The bearing stratum must be adequately compacted to avoid adverse settlements.

d. Typical Illustration



d. Earth Stabilization Columns. Many methods are available for forming compression reinforcement elements (see DM-7.3, Chapter 2) including:

(1) Mixed-In-Place Piles. A mixed-in-place soil-cement or soil-lime pile.

(2) Vibro-Replacement Stone Columns. A vibroflot or other device is used to make a cylindrical, vertical hole which is filled with compacted gravel or crushed rock.

(3) Grouted Stone Columns. This is similar to the above but includes filling voids with bentonite-cement or water-sand-bentonite cement mixtures.

(4) Concrete Vibro Columns. Similar to stone columns but concrete introduced instead of gravel.

Section 3. BEARING CAPACITY AND SETTLEMENT

1. DESIGN PROCEDURES. The design of a deep foundation system should include the following steps:

(1) Evaluate the subsurface conditions.

(2) Review the foundation requirements including design loads and allowable settlement or deflection.

(3) Evaluate the anticipated construction conditions and procedures.

(4) Incorporate local experience and practices.

(5) Select appropriate foundation type(s) based on the above items, costs, and comments on Tables 1 and 2.

(6) Determine the allowable axial foundation design loads based on an evaluation of ultimate foundation capacity including reductions for group action or downdrag if applicable, anticipated settlement and local requirements and practices.

The axial load capacity of deep foundations is a function of the structural capacity of the load carrying member (with appropriate reduction for column action) and the soil load carrying capacity. Usually, the latter consideration controls design. The methods available for evaluating the ultimate axial load capacity are listed below. Some or all of these should be considered by the design engineer as appropriate.

(a) Static analysis utilizing soil strength.

(b) Empirical analysis utilizing standard field soil tests.

(c) Building code requirements and local experience.

(d) Full-scale load tests.

(e) Dynamic driving resistance.

(7) Determine design and construction requirements, and incorporate the requirements into construction specifications.

Inspection of foundation construction should be considered an integral part of the design procedures. Perform a pile test program as required. The pile test can also be used as a design tool in item (6).

2. BEARING CAPACITY OF SINGLE PILE

a. Allowable Stresses. See Table 1 for allowable stresses within the pile and quality requirements for pile materials. Allowable stresses should be reduced for column action where the pile extends above firm ground, i.e. through water and very soft bottom sediments.

b. Soil Support. The soil must be capable of supporting the element when it is in compression, tension, and subject to lateral forces. The soil support can be computed from soil strength data, determined by load tests, and/or estimated from driving resistance. These determinations should include the following stages:

(1) Design Stage. Compute required pile lengths from soil strength data to determine bidding length and pile type.

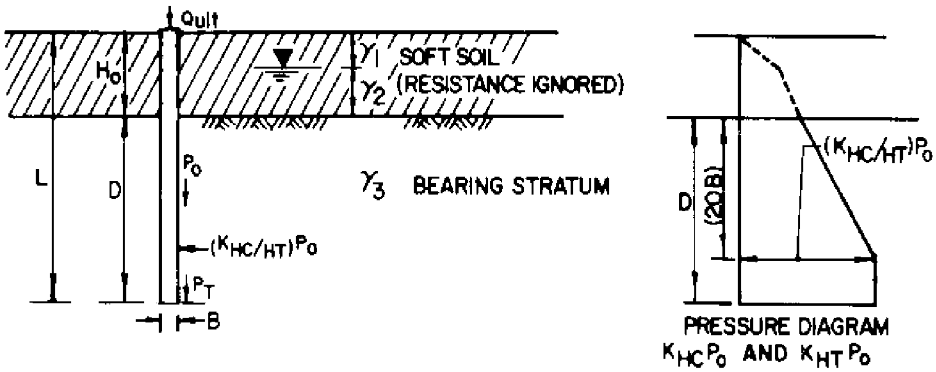
(2) Early in Construction Stage. Drive test piles at selected locations. For small projects where performance of nearby pile foundations is known, base design length and load capacity on knowledge of the soil profile, nearby pile performance, and driving resistance of test piles. On large projects where little experience is available, perform load tests on selected piles and interpret the results as shown in Figure 7.

(3) Throughout Construction Stage. Record driving resistance of all piles for comparison with test piles and to insure against local weak subsurface formations. Record also the type and condition of cushioning material used in the pile hammer.

c. Theoretical Load Capacity. See Figure 1 for analysis of ultimate load carrying capacity of single piles in homogeneous granular soils; for pile in homogeneous cohesive soil see Figure 2 (upper panel right, Reference 2, The Bearing Capacity of Clays, by Skempton; remainder of figure, Reference 3, The Adhesion of Piles Driven in Clay Soils, by Tomlinson).

(1) Compression Load Capacity. Compression load capacity equals end-bearing capacity, plus frictional capacity on perimeter surface.

(2) Pullout Capacity. Pullout capacity equals the frictional force on the perimeter surface of the pile or pier.



(A) ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = P_T N_q A_T + \sum_{H=H_o}^{H=H_o+D} (K_{HC})(P_o)(\tan\delta)(S)$$

WHERE Q_{ult} = ULTIMATE LOAD CAPACITY IN COMPRESSION

P_T = EFFECTIVE VERTICAL STRESS AT PILE TIP (SEE NOTE 1)

N_q = BEARING CAPACITY FACTOR (SEE TABLE, FIGURE 1 CONTINUED)

A_T = AREA OF PILE TIP

K_{HC} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN COMPRESSION.

P_o = EFFECTIVE VERTICAL STRESS OVER LENGTH OF EMBEDMENT, D (SEE NOTE 1)

δ = FRICTION ANGLE BETWEEN PILE AND SOIL (SEE TABLE, FIGURE 1 CONTINUED)

S = SURFACE AREA OF PILE PER UNIT LENGTH

FOR CALCULATING Q_{all}, USE F_S OF 2 FOR TEMPORARY LOADS, 3 FOR PERMANENT LOADS. (SEE NOTE 2)

(B) ULTIMATE LOAD CAPACITY IN TENSION

$$T_{ult} = \sum_{H=H_o}^{H=H_o+D} (K_{HT})(P_o)(\tan\delta)(S)(H)$$

WHERE: T_{ult} = ULTIMATE LOAD CAPACITY IN TENSION, PULLOUT

K_{HT} = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON SIDE OF ELEMENT WHEN ELEMENT IS IN TENSION

FOR CALCULATING T_{all}, USE $F_S = 3$ ON T_{ult} PLUS THE WEIGHT OF THE PILE (W_P), THUS $T_{all} = \frac{T_{ult}}{3} + W_P$ (SEE NOTE 2)

NOTE-1: EXPERIMENTAL AND FIELD EVIDENCE INDICATE THAT BEARING PRESSURE AND SKIN FRICTION INCREASE WITH VERTICAL EFFECTIVE STRESS P_o UP TO A LIMITING DEPTH OF EMBEDMENT, DEPENDING ON THE RELATIVE DENSITY OF THE GRANULAR SOIL AND POSITION OF THE WATER TABLE. BEYOND THIS LIMITING DEPTH (10B ± TO 40B ±) THERE IS VERY LITTLE INCREASE IN END BEARING, AND INCREASE IN SIDE FRICTION IS DIRECTLY PROPORTIONAL TO THE SURFACE AREA OF THE PILE. THEREFORE, IF D IS GREATER THAN 20 B, LIMIT P_o AT THE PILE TIP TO THAT VALUE CORRESPONDING TO D = 20B.

NOTE-2: IF BUILDING LOADS AND SUBSURFACE CONDITION ARE WELL DOCUMENTED IN THE OPINION OF THE ENGINEER, A LESSER FACTOR OF SAFETY CAN BE USED BUT NOT LESS THAN 2.0 PROVIDED PILE CAPACITY IS VERIFIED BY LOAD TEST AND SETTLEMENTS ARE ACCEPTABLE.

FIGURE 1
Load Carrying Capacity of Single Pile in Granular Soils

BEARING CAPACITY FACTORS - Nq

[phi][*] (DEGREES)	26	28	30	31	32	33	34	35	36	37	38	39	40
N_q (DRIVEN PILE DISPLACEMENT)	10	15	21	24	29	35	42	50	62	77	86	120	145
N_q [**] (DRILLED PIERS)	5	8	10	12	14	17	21	25	30	38	43	60	72

EARTH PRESSURE COEFFICIENTS K_{HC} AND K_{HT}

PILE TYPE	K_{HC}	K_{HT}
DRIVEN SINGLE H-PILE	0.5 - 1.0	0.3 - 0.5
DRIVEN SINGLE DISPLACEMENT PILE	1.0 - 1.5	0.6 - 1.0
DRIVEN SINGLE DISPLACEMENT TAPERED PILE	1.5 - 2.0	1.0 - 1.3
DRIVEN JETTED PILE	0.4 - 0.9	0.3 - 0.6
DRILLED PILE (LESS THAN 24" DIAMETER)	0.7	0.4

FRICTION ANGLE - [delta]

PILE TYPE	[delta]
STEEL	20 deg.
CONCRETE	3/4 [phi]
TIMBER	3/4 [phi]

[*] LIMIT [phi] TO 280 IF JETTING IS USED

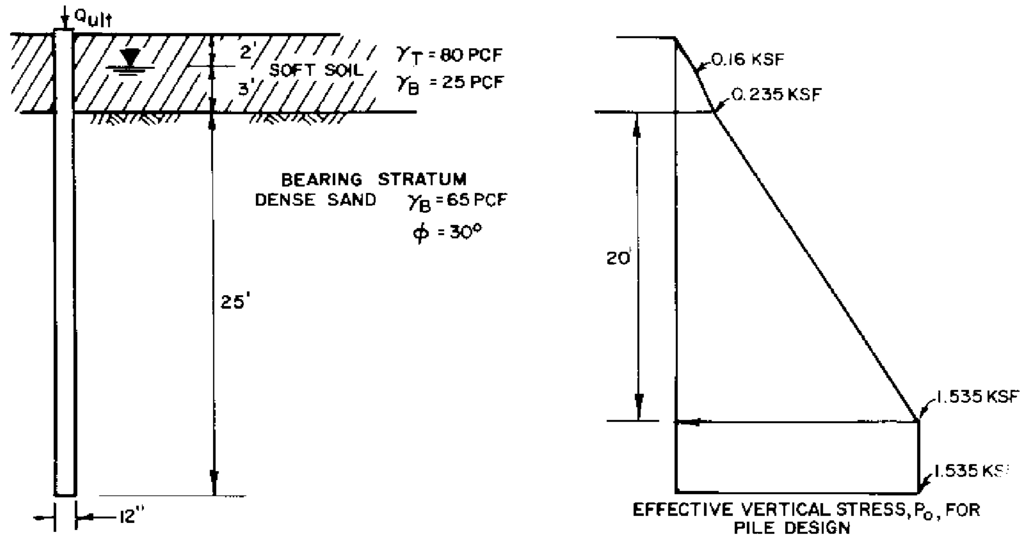
[**] (A) IN CASE A BAILER OR GRAB BUCKET IS USED BELOW GROUND WATER TABLE, CALCULATE END BEARING BASED ON [phi] NOT EXCEEDING 28 deg.

(B) FOR PIERS GREATER THAN 24-INCH DIAMETER, SETTLEMENT RATHER THAN BEARING

*   CAPACITY USUALLY CONTROLS THE DESIGN. FOR ESTIMATING SETTLEMENT, TAKE 50% *
*   OF THE SETTLEMENT FOR AN EQUIVALENT FOOTING RESTING ON THE SURFACE OF   *
*   COMPARABLE GRANULAR SOILS. (CHAPTER 5, DM-7.1).   *

.))-

FIGURE 1 (continued)
Load Carrying Capacity of Single Pile in Granular Soils



FOR A 12" DIAMETER CLOSED END, DRIVEN PIPE PILE, CONCRETE FILLED, FIND Q_{all} AND T_{all} FOR A 30 FOOT LONG PILE.

P_o MAX OCCURS AT 20B, OR 20' INTO BEARING STRATUM.

$\phi = 30°$, $N_q = 21$

$K_{HC} = 1.5$, $\delta = 20°$

$K_{HT} = 1.0$

$A_T = \pi \times 0.5^2 = 0.78$ SF

CIRCUM. AREA / lf = 1 X π = 3.14 SF / lf

$$Q_{ult} = 1.535 \times 21 \times 0.78 + \left[\left(1.5 \times \left(\frac{0.235 + 1.535}{2}\right) \times \text{TAN } 20 \times 20 \times 3.14\right) + \left(1.5 \times 1.535 \times \text{TAN } 20 \times 5 \times 3.14\right) \right]$$

$$= 25.14 + \left[30.34 + 13.16\right]$$

$$= 68.64 \text{ K}$$

FOR $F_S = 3$, $Q_{all} = \dfrac{68.64}{3} = 22.9$ K

$$T_{ult} = 1.0 \times \left(\frac{0.235 + 1.535}{2}\right) \times \text{TAN } 20 \times 20 \times 3.14 + 1.0 \times 1.535 \times \text{TAN } 20 \times 5 \times 3.14$$

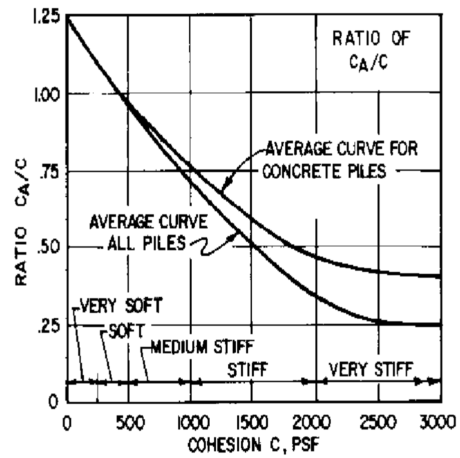
$$= 20.23 + 8.77$$

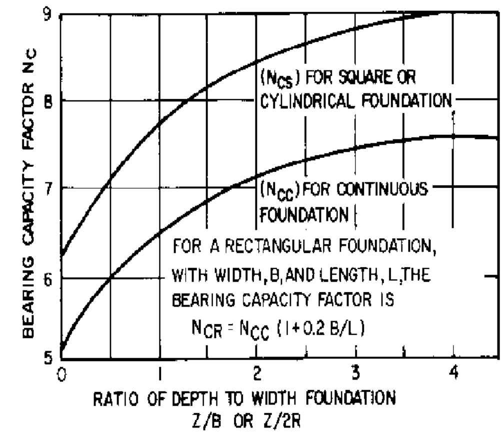
$$= 29.0 \text{ K}$$

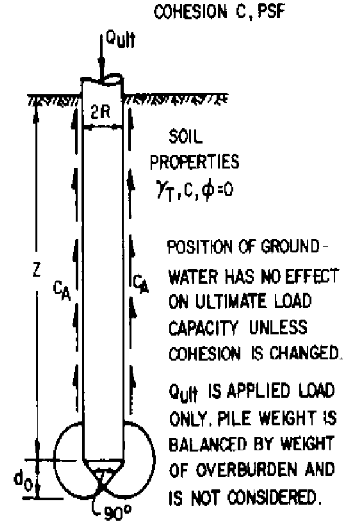
$W_P \approx .117$ K/lf X 30' = 3.5 K

$$T_{all} = \frac{29.0}{3} + 3.5 = 13.2 \text{K}$$

FIGURE 1 (continued)
Load Carrying Capacity of Single Pile in Granular Soils







ULTIMATE LOAD CAPACITY IN COMPRESSION

$$Q_{ult} = C\,\begin{pmatrix}N_{CS}\\ N_{CC}\end{pmatrix}\pi R^2 + C_A\,2\pi RZ$$

RECOMMENDED VALUES OF ADHESION

PILE TYPE	CONSISTENCY OF SOIL	COHESION, C PSF	ADHESION, C_A PSF
TIMBER AND CONCRETE	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 480
	MED. STIFF	500 - 1000	480 - 750
	STIFF	1000 - 2000	750 - 950
	VERY STIFF	2000 - 4000	950 - 1300
STEEL	VERY SOFT	0 - 250	0 - 250
	SOFT	250 - 500	250 - 460
	MED. STIFF	500 - 1000	460 - 700
	STIFF	1000 - 2000	700 - 720
	VERY STIFF	2000 - 4000	720 - 750

ULTIMATE LOAD CAPACITY IN TENSION

$T_{ult} = C_A\, 2\pi RZ$

T_{ult} UNDER SUSTAINED LOAD MAY BE LIMITED BY OTHER FACTORS, SEE TEXT.

FIGURE 2
Ultimate Load Capacity of Single Pile or Pier in Cohesive Soils

(3) Drilled Piers. For drilled piers greater than 24 inches in diameter settlement rather than bearing capacity may control. A reduced end bearing resistance may result from entrapment of bentonite slurry if used to maintain an open excavation to the pier's tip. Bells, or enlarged bases, are usually not stable in granular soils.

(4) Piles and Drilled Piers in Cohesive Soils. See Figure 2 and Table 3. Experience demonstrates that pile driving permanently alters surface adhesion of clays having a shear strength greater than 500 psf (see Figure 2). In softer clays the remolded material consolidates with time, regaining adhesion approximately equal to original strength. Shear strength for point-bearing resistance is essentially unchanged by pile driving. For drilled piers, use Table 3 from Reference 4, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures, by lie Departments of Army and Air Force, for determining side friction. Ultimate resistance to pullout cannot exceed the total resistance of reduced adhesion acting over the pile surface or the effective weight of the soil mass which is available to react against pullout. The allowable sustained pullout load usually is limited by the tendency for the pile to move upward gradually while mobilizing an adhesion less than the failure value.

Adhesion factors in Figure 2 may be very conservative for evaluating piles driven into stiff but normally consolidated clays. Available data suggests that for piles driven into normally to slightly overconsolidated clays, the side friction is about 0.25 to 0.4 times the effective overburden.

(5) Piles Penetrating Multi-layered Soil Profile. Where piles penetrate several different strata, a simple approach is to add supporting capacity of the individual layers, except where a soft layer may consolidate and relieve load or cause drag on the pile. For further guidance on bearing capacity when a pile penetrates layered soil and terminates in granular strata see Reference 5, Ultimate Bearing Capacity of Foundations on Layered Soils Under Inclined Loads, by Meyer off and Hanna, which consides the ultimate bearing capacity of a deep member in sand underlying a clay layer and for the case of a sand bearing stratum overlying a weak clay layer.

(6) Pile Buckling. For fully embedded piles, buckling usually is not a problem. For a fully embedded, free headed pile with length equal to or greater than 4T, the critical load for buckling is as follows (after Reference 6, Design of Pile Foundations, by Vesic):

P+crit, = 0.78 T.3-f for L>/= 4T

where: P+crit, = critical load for buckling

f = coefficient of variation of lateral subgrade reaction (see Figure 10)

T = relative stiffness factor (see Figure 10)

L = length of pile.

TABLE 3

Design Parameters for Side Friction for Drilled Piers in Cohesive Soils

Design Category	Side Resistance		Remarks
	C_A/C	Limit on side shear - tsf	
A. Straight-sided shafts in either homogeneous or layered soil with no soil of exceptional stiffness below the base			
1. Shafts installed dry or by the slurry displacement method	0.6	2.0	
2. Shafts installed with drilling mud along some portion of the hole with possible mud entrapment	0.3(a)	0.5(a)	(a) C_A/C may be increased to 0.6 and side shear increased to 2.0 tsf for segments drilled dry
B. Belled shafts in either homogeneous or layered clays with no soil of exceptional stiffness below the base			
1. Shafts installed dry or by the slurry displacement methods	0.3	0.5	
2. Shafts installed with drilling mud along some portion of the hole with possible mud entrapment	0.15(b)	0.3(b)	(b) C_A/C may be increased to 0.3 and side shear increased to 0.5 tsf for segments drilled dry

TABLE 3 (continued)
Design Parameters for Side Friction for Drilled Piers in Cohesive Soils

Design Category	Side Resistance		Remarks
	C_A/C	Limit on side shear - tsf	
C. Straight-sided shafts with base resting on soil significantly stiffer than soil around stem	0	0	
D. Belled shafts with base resting on soil significantly stiffer than soil around stem	0	0	

Note: In calculating load capacity, exclude: (1) top 5 feet of drilled shaft: (2) periphery of bell: and (3) bottom 5 feet of straight shaft and bottom 5 feet of stem of shaft above bell.

For piles with the head fixed against rotation and translation, increase P_{crit} by 13%. If the pile head is pinned (i.e. prevented from translation but free to rotate), increase P_{crit} by 62%.

For a partially embedded pile, assume a free standing column fixed at depth 1.8T below the soil surface. Compute the critical buckling load by methods of structural analysis. For such piles compute allowable pile stresses to avoid buckling. For the case where the coefficient of lateral subgrade reaction (K_h) of the embedment soil is constant with depth, calculate the depth of fixity as $1.4 \sqrt[4]{\frac{EI}{K_h B}}$, where EI is the flexural rigidity of the pile, B is pile width (diameter) and K_h is defined in the units of Force/Length^3. Buckling for a fully embedded length of other pile types does not control pile stress. For further guidance see Reference 6.

d. Empirical Bearing Capacity. Results from the Standard Penetration Test, Static Cone penetrometer (Dutch Cone with friction sleeve), and Pressuremeter have been correlated with model and full scale field tests on piles and deep foundations so that empirical expressions are available to estimate foundation capacities.

(1) Standard Penetration. Use of the Standard Penetration Test to predict capacities of deep foundations should be limited to granular soils and must be considered a crude estimate.

Tip Resistance of driven piles (after Reference 7, Bearing Capacity and Settlement of Pile Foundations, by Meyerhof):

$$q_{ult} = \frac{0.4 \bar{N} D}{B} \leq q$$

where: $\bar{N} = C_N - N$

N = standard penetration resistance (blow/ft) near pile tip

$$C_N = 0.77 \log_{10} \frac{20}{p} \quad (\text{for } p \geq 0.25 \text{ TSF})$$

p = effective overburden stress at pile tip (TSF)

q_{ult} = ultimate point resistance of driven pile (TSF)

$\bar{N}$ = average corrected Standard Penetration Resistance near pile tip (blows/ft)

D = depth driven into granular bearing stratum (ft)

B = width or diameter of pile tip (feet)

q_l = limiting point resistance (TSF), equal to 4N for sand and 3N for non-plastic silt.

For drilled piers, use 1/3 times q_{ult}, computed from the above expression.

Use a factor of safety of 3 to compute allowable tip resistance.

Skin Friction of driven piles:

$$f_s = \frac{N}{50} \leq f_l$$

where: N = average standard penetration along pile shaft

f_s = ultimate skin friction for driven pile (TSF)

f_l = limiting skin friction (for driven pile, f_l = (TSF)

Use factor of safety of 3 for allowable skin friction.

For driven piles tapered more than 1 percent, use 1.5 times above expression.

For drilled piers, use 50 percent of above expression

(2) The Cone Penetrometer. The Cone Penetrometer provides useful information as a "model pile" and is best suited for loose to dense sands and silts. Penetrometer results are not considered accurate for very dense sands or deposits with gravel.

Point Resistance:

$$q_{ult} = q_c$$

where: q_{ult} = ultimate tip resistance for driven pile

q_c = cone penetration resistance

Depth of penetration to granular bearing stratum is at least 10 times the pile tip width.

Shaft Resistance:

$$f_{ult} = f_c$$

where: f_{ult} = ultimate shaft friction of driven cylindrical pile

f_c = unit resistance of local friction sleeve of static penetrometer

Use factor of safety of 3 for allowable skin friction.

For drilled piers in cohesionless soil, use 1/2 of f_{ult}, or q_{ult}, based on the above expressions for driven piles.

(3) Pressuremeter. Results from pressuremeter tests can be used to estimate design capacity of deep foundation elements. See Reference 8, The Pressuremeter and Foundation Engineering, by Baguelin, et al., or Reference 9, Canadian Foundation Engineering Manual, by the Canadian Geotechnical Society, for details of design correlation.

The pressuremeter method is useful in soft rock, weathered or closely jointed rock, granular soils, and very stiff cohesive soils. Results are generally not suitable in soft clays because of the disturbance during drilling. The self-boring pressuremeter is designed to reduce this problem.

e. Bearing Capacity from Dynamic Driving Resistance.

(1) General. The ultimate capacity of a pile may be estimated on the basis of driving resistance during installation of the pile. The results are not always reliable, and may over-predict or grossly under-predict pile capacities, and therefore should be used with caution. Use must be supported by local experience or testing. Dynamic resistance based on the wave equation analysis is a more rational approach to calculating pile capacities.

(2) Pile Driving Formulas:

(a) General. Because of the uncertainties of the dynamics of pile driving, the use of formulas more elaborate than those in Table 4 is not warranted. A minimum of three test piles should be driven for each installation, with more tests if subsurface conditions are erratic.

(b) Control During Construction. The embedment of piles should be controlled by specifying a minimum tip elevation on the basis of the subsurface profile and driving tests or load tests, if available, and also by requiring that the piles be driven beyond the specified elevation until the driving resistance equals or exceeds the value established as necessary from the results of the test piles. However, if the pile penetration consistently overruns the anticipated depth, the basis for the specified depth and driving resistance should be reviewed.

(c) Formulas. Dynamic pile driving formulas should not be used as criteria for establishing load capacity without correlation with the results of an adequate program of soil exploration. For critical structures and where local experience is limited, or where unfamiliar pile types or equipment are being used, load tests should be performed.

(3) Wave Equation Analysis. The wave equation analysis is based on the theory of one dimensional wave propagation. For the analysis the pile is divided into a series of masses connected by springs which characterize the pile stiffness, and dashpots which simulate the damping below the pile tip and along pile embedded length.

This method was first put into practical form in 1962 Reference 10, Pile Driving by the Wave Equation, by Smith). The wave equation analysis provides a means of evaluating the suitability of the pile stiffness to transmit driving energy to the tip to achieve pile penetration, as well as the ability of pile section to withstand driving stresses without damage. The results of the analysis can be interpreted to give the following:

TABLE 4

Application of Pile Driving Resistance Formulas

BASIC PILE DRIVING FORMULAS (SEE COMMENT IN SECTION 2)

FOR DROP HAMMER	FOR SINGLE - ACTING HAMMER	FOR DOUBLE - ACTING DIFFERENTIAL HAMMER
$Q_{all} = \frac{2WH}{S+1}$	$Q_{all} = \frac{2WH}{S+0.1}$ USE WHEN DRIVEN WEIGHTS ARE SMALLER THAN STRIKING WEIGHTS.	$Q_{all} = \frac{2E}{S+0.1}$ USE WHEN DRIVEN WEIGHTS ARE SMALLER THEN STRIKING WEIGHTS.
	$Q_{all} = \frac{2WH}{S+0.1\frac{W_D}{W_S}}$ USE WHEN DRIVEN WEIGHTS ARE LARGER THAN STRIKING WEIGHTS.	$Q_{all} = \frac{2E}{S+0.1\cdot\frac{W_D}{W_S}}$ USE WHEN DRIVEN WEIGHTS ARE LARGER THAN STRIKING WEIGHTS.

Q_{all} = ALLOWABLE PILE LOAD IN POUNDS.
W = WEIGHT OF STRIKING PARTS OF HAMMER IN POUNDS.
H = THE EFFECTIVE HEIGHT OF FALL IN FEET.
E = THE ACTUAL ENERGY DELIVERED BY HAMMER PER BLOW IN FOOT-POUNDS.
S = AVERAGE NET PENETRATION IN INCHES PER BLOW FOR THE LAST 6 IN. OF DRIVING.
W_D = DRIVEN WEIGHTS
W_S = WEIGHTS OF STRIKING PARTS

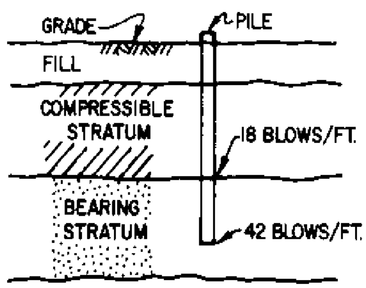
NOTE: RATIO OF DRIVEN WEIGHTS TO STRIKING WEIGHTS SHOULD NOT EXCEED 3.

MODIFICATIONS OF BASIC PILE DRIVING FORMULAS

A. FOR PILES DRIVEN TO AND SEATED IN ROCK AS HIGH CAPACITY END-BEARING PILES:
DRIVE TO REFUSAL (APPROXIMATELY 4 TO 5 BLOWS FOR THE LAST QUARTER INCH OF DRIVING).
REDRIVE OPEN END PIPE PILES REPEATEDLY UNTIL RESISTANCE FOR REFUSAL IS REACHED WITHIN 1 IN. OF ADDITIONAL PENETRATION.

B. PILES DRIVEN THROUGH STIFF COMPRESSIBLE MATERIALS UNSUITABLE FOR PILE BEARING TO AN UNDERLYING BEARING STRATUM:

ADD BLOWS ATTAINED BEFORE REACHING BEARING STRATUM TO REQUIRED BLOWS ATTAINED IN BEARING STRATUM (SEE EXAMPLE).



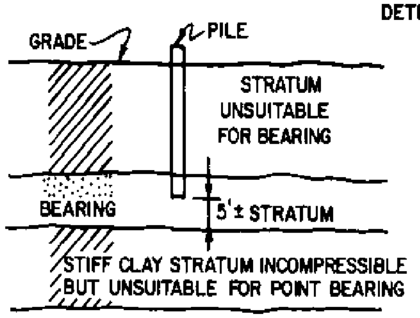
EXAMPLE: REQUIRED LOAD CAPACITY OF PILE Q_{all} = 25 TONS
HAMMER ENERGY E = 15,000 FT.-LB.

$\frac{W_d}{W_s} < 1$

PENETRATION(S) AS PER BASIC FORMULA = 1/2" OR 2 BLOWS PER INCH (24 BLOWS/FT).

REQUIRED BLOWS FOR PILE 24 + 18 = 42 BLOWS/FT.

C. PILES DRIVEN INTO LIMITED THIN BEARING STRATUM, DRIVE TO PREDETERMINED TIP ELEVATION. DETERMINE ALLOWABLE LOAD BY LOAD TEST.



(a) Equipment compatibility: appropriate hammer size and cushion.

(b) Driving stresses: plots of stress vs. set can be made to evaluate the potential for pile overstress.

(c) Pile capacity: plot of ultimate pile capacity vs. set can be developed.

The soil is modeled by approximating the static resistance (quake), the viscous resistance (damping), and the distribution of the soil resistance along the pile. The assigned parameter for springs and dashpots cannot be related to routinely measured soil parameters which constitutes the major draw back of the wave equation analysis. The input for the driving system is provided by the anticipated hammer performance, coefficient of restitution of the cushion, and stiffness of the pile. Computer programs are available to perform the lengthy calculations.

(4) Case Method. The wave equation analysis can be used in conjunction with field measurements by using the Case Method (Reference 11, Soil Resistance Predictions from Pile Dynamics, by Rausche, et al.). This procedure electronically measures the acceleration and strain near the top of the pile, and by using the wave equation analysis estimates the static soil resistance for each blow of the hammer. Energy transferred to the pile is computed by integrating the product of force and velocity. A distribution of the soil resistance along the pile length is assumed and the wave equation analysis is performed. The assumed soil strength parameters are checked against the measured force at the pile top and these are then adjusted to result in an improved match between the analytical and measured pile force at the top.

3. BEARING CAPACITY OF PILE GROUPS.

a. General. The bearing capacity of pile groups in soils is normally less than the sum of individual piles in the group and must be considered in design. Group efficiency is a term used for the ratio of the capacity of a pile group to the sum of the capacities of single piles at the same depth in the same soil deposit. In evaluating the performance of pile groups in compression, settlement is a major consideration. Expressions for estimating uplift resistance of pile groups are included in this section.

b. Group Capacity in Rock. The group capacity of piles installed to rock is the number of members times the individual capacity of each member. Block failure is a consideration only if foundations are on a sloping rock formation, and sliding may occur along unfavorable dipping weak planes. The possibility of such an occurrence must be evaluated from the site geology and field exploration.

c. Group Capacity in Granular Soil. Piles driven into cohesionless soil in a group configuration act as individual piles if the spacing is greater than 7 times the average pile diameter. They act as a group at close spacings. Center to center spacing of adjacent piles in a group should be at least two times the butt diameter.

Block failure of a pile group in granular soils is not a design consideration provided each individual pile has an adequate factor of safety against bearing failure and the cohesionless soil is not underlain by a weaker deposit. In loose sand and/or gravel deposits, the load carrying capacity of an individual pile may be greater in the group than single because of densification during driving. This increased efficiency should be included in design with caution, and only where demonstrated by field experience or tests.

The ultimate capacity of a pile group founded in dense cohesionless soil of limited thickness underlain by a weak deposit is the smaller of:

(1) sum of the single pile capacities

(2) block failure of a pier equivalent in size to the piles and enclosed soil mass, punching through the dense deposit into the underlying weak deposit (Reference 12, Ultimate Bearing Capacity of Footings on Sand Layer Overlying Clay, by Meyerhof).

d. Group Capacity in Cohesive Soil. Estimate the group capacity using the method in Figure 3 (upper panel, Reference 13, Experiments with Model Piles in Groups, by Whitaker).

e. Uplift Resistance of Groups.

(1) Granular Soil. Ultimate uplift resistance of pile group is lesser of:

(a) Sum of skin friction on the piles in the group (no reduction for tapered piles), use a factor of safety of 3.0.

(b) Effective weight of block of soil within the group and within a 4 vertical on 1 horizontal wedge extending up from pile tips - weight of piles assumed equal to volume of soil they displace. Factor of safety should be unity.

(2) Cohesive Soil. Ultimate uplift resistance of pile group is the lesser of:

(a) Sum of skin friction on the piles in the group

(b) T+u, = L (B + A) C + W+p,

where: T+u, = ultimate uplift resistance of pile group

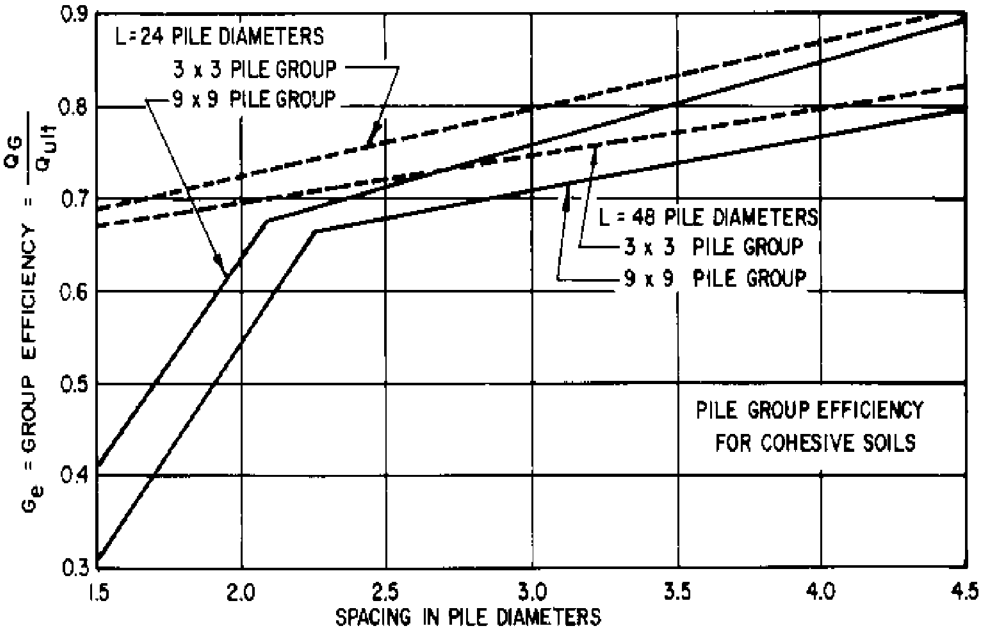
A = length of group

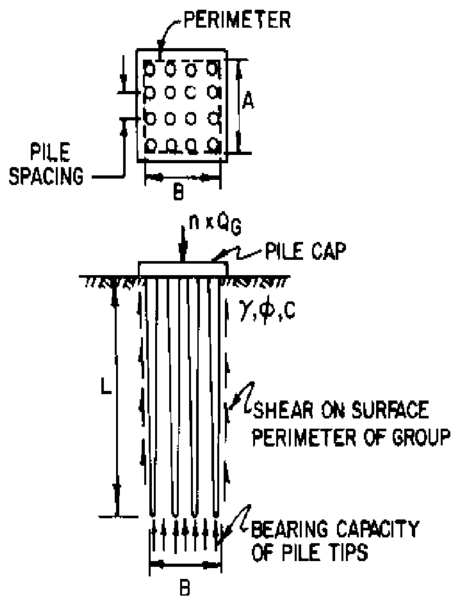
B = width of group

L = depth of soil block below pile cap

C = average undrained strength of soil around the sides of the group

W+p, = weight of piles, pile cap, and block of soil enclosed by the piles.





DEFINITIONS

Q_G = ULTIMATE LOAD CAPACITY OF PILE IN GROUP

Q_{ult} = ULTIMATE LOAD CAPACITY OF ISOLATED PILE

n = NUMBER OF PILES IN GROUP

$G_e = \frac{Q_G}{Q_{ult}}$ FOR COHESIVE SOILS

2R = PILE DIAMETER

Q_G AND Q_{ult} ARE APPLIED LOADS ONLY. WEIGHT OF PILES AND ENCLOSED SOIL IS BALANCED BY WEIGHT OF OVERBURDEN AND IS NOT CONSIDERED.

BEARING CAPACITY OF PILE GROUP

$Q_{ult} = (cN_c)\pi R^2 + C_A 2\pi RL$ (OBTAIN C_A AND N_c FROM FIGURE 2)

ULTIMATE LOAD OF GROUP = $nQ_G = G_e n Q_{ult}$

FIGURE 3
Bearing Capacity of Pile Groups in Cohesive Soils

Factors of Safety: 2 for short-term loads, 3 for sustained uplifting loading.

4. SETTLEMENTS OF PILE FOUNDATIONS

a. Single Pile. The settlement at the top of pile can be broken down into three components (after Reference 6).

(1) Settlement due to axial deformation of pile shaft; Ws

$$W_s = (Q_p + \alpha_s Q_s)\frac{L}{AE_p}$$

where: Q_p = point load transmitted to the pile tip in the working stress range.

Q_s = shaft friction load transmitted by the pile in the working stress range (in force units)

α_S = 0.5 for parabolic or uniform distribution of shaft friction

0.67 for triangular distribution of shaft friction starting from zero friction at pile head to a maximum value at pile point

0.33 for triangular distribution of shaft friction starting from a maximum at pile head to zero at the pile point.

L = pile length

A = pile cross sectional area

E_p = modulus of elasticity of the pile

(2) Settlement of pile point caused by load transmitted at the point W_{pp}:

$$W_{pp} = \frac{C_p\,Q_p}{Bq_o}$$

where: C_p = empirical coefficient depending on soil type and method of construction, see Table 5

B = pile diameter

q_o = ultimate end bearing capacity

(3) Settlement of pile points caused by load transmitted along the pile shaft, W_{ps};

$$W_{ps} = \frac{C_s Q_s}{Dq_o}$$

TABLE 5
Typical[*] Values of Coefficient C+p, for Estimating Settlement of a Single Pile

Soil Type	Driven Piles	Bored Piles
Sand (dense to loose)	0.02 to 0.04	0.09 to 0.18
Clay (stiff to soft)	0.02 to 0.03	0.03 to 0.06
Silt (dense to loose)	0.03 to 0.05	0.09 to 0.12

[*] Bearing stratum under pile tip assumed to extend at least 10 pile diameters below tip and soil below tip is of comparable or higher stiffness.

where: $C_s = (0.93 + 0.16\, D/B)\, C_p$

D = embedded length

(4) Total settlement of a single pile, W_o:

$$W_o = W_s + W_{pp} + W_{ps}$$

b. Settlement of Pile Group in Granular Soils. Compute the group settlement W_g, based on (after Reference 6):

$$W_g = W_o \sqrt{\bar{B}/B}$$

where: $\bar{B}$ = the smallest dimension of pile group

B = diameter of individual pile

W_o = Settlement of a single pile estimated or determined from load tests

c. Settlement of Pile Groups in Saturated Cohesive Soils. Compute the group settlement as shown in Figure 4.

d. Limitations. The above analyses may be used to estimate settlement, however, settlement estimated from the results of load tests are generally considered more accurate and reliable.

5. NEGATIVE SKIN FRICTION.

a. General. Deep foundation elements installed through compressible material can experience "downdrag" forces or negative skin friction along the shaft which results from downward movement of adjacent soil relative to the pile. Negative skin friction results primarily from consolidation of a soft deposit caused by dewatering or the placement of fill.

Negative skin friction is particularly severe on batter pile installations because the force of subsiding soil is large on the outer side of the batter pile and soil settles away from the inner side of the pile. This can result in bending of the pile. Batter pile installations should be avoided where negative skin friction is expected to develop.

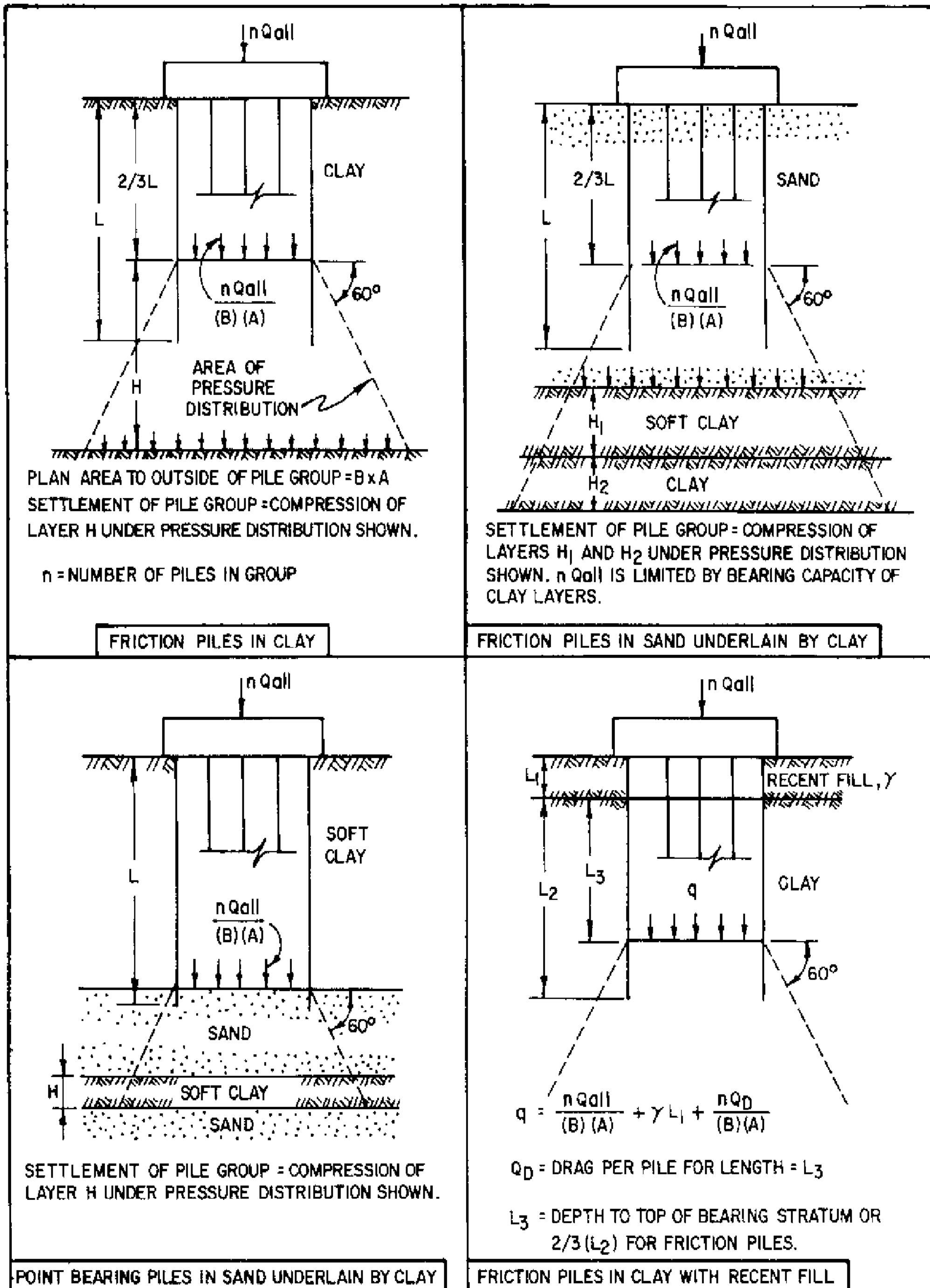
b. Distribution of Negative Skin Friction on Single Pile. The distribution and magnitude of negative skin friction along a pile shaft depends on:

(1) relative movement between compressible soil and pile shaft;

(2) relative movement between upper fill and pile shaft;

(3) elastic compression of pile under working load;

(4) rate of consolidation of compressible soils.



NOTES: 1. PLAN AREA TO OUTSIDE OF PILE GROUP = B x A.
2. FOR RELATIVELY RIGID PILE CAP, PRESSURE DISTRIBUTION IS ASSUMED TO VARY WITH DEPTH AS SHOWN.
3. FOR FLEXIBLE SLAB OR GROUP OF SMALL SEPARATE CAPS, COMPUTE PRESSURES BY ELASTIC SOLUTIONS (DM-7.1 CHAPTER 4) FOR LOAD APPLIED AT LEVEL SHOWN.
4. COMPUTE SETTLEMENTS BY METHODS OF DM 7.1 CHAPTER 5.

FIGURE 4
Settlement of Pile Groups

Negative skin friction develops along that portion of the pile shaft where settlement of the adjacent soil exceeds the downward displacement of the shaft. The "neutral point" is that point of no relative movement between the pile and adjacent soil. Below this point, skin friction acts to support pile loads. The ratio of the depth of the neutral point to the length of the pile in compressible strata may be roughly approximated as 0.75. The position of the neutral point can be estimated by a trial and error procedure which compares the settlement of the soil to the displacement of adjacent sections of the pile. (For further guidance see Reference 14, Pile Design and Construction Practice, by Tomlinson.)

Observations indicate that a relative downward movement of 0.6 inch is expected to be sufficient to mobilize full negative skin friction (Reference 6).

c. Magnitude of Negative Skin Friction on Single Pile. The peak negative skin friction in granular soils and cohesive soils is determined as for positive skin friction.

The peak unit negative skin friction can also be estimated from (after Reference 15, Prediction of Downdrag Load at the Cutler Circle Bridge, by Garlanger):

$$f_n = \beta P_o$$

where: f_n = unit negative skin friction (to be multiplied by area of shaft in zone of subsiding soil relative to pile)

P_o = effective vertical stress

β = empirical factor from full scale tests

Soil	β
Clay	0.20 - 0.25
Silt	0.25 - 0.35
Sand	0.35 - 0.50

d. Safety Factor for Negative Skin Friction. Since negative skin friction is usually estimate on the safe side, the factor of safety associated with this load is usually unity. Thus:

$$Q_{all} = \frac{Q_{ult}}{F_s} - P_n$$

where: Q_{all} = allowable pile load

Q_{ult} = ultimate pile load

F_s = factor of safety

P_n = ultimate negative skin friction load

For further discussion of factor of safety in design including transient loads, see Reference 16, Downdrag on Piles Due to Negative Skin Friction, by Fellenius.

e. Negative Skin Friction on Pile Groups. The negative skin friction on a pile group does not usually exceed the total weight of fill and/or compressible soil enclosed by the piles in the group. For the case of recent fill underlain by a compressible deposit over the bearing stratum:

$$P_{total} \leq W + (B)(L)(\gamma_1 D_1 + \gamma_2 D_2)$$

where:

P_{total} = total load on pile group

W = working load on pile group

B = width of pile group

L = length of pile group

γ_1, γ_2 = effective unit weight of fill and underlying compressible soil respectively

D_1, D_2 = depth over which fill and compressible soil is moving downward relative to the piles

f. Reduction of Negative Skin Friction. Several methods have been developed to reduce the expected negative skin friction on deep foundations. These include:

(a) Use of slender piles, such as H-sections, to reduce shaft area subject to drag.

(b) Predrilled oversized hole through compressible material prior to insertion of pile (resulting annular space filled with bentonite slurry or vermiculite)

(c) Provide casing or sleeve around pile to prevent direct contact with settling soil.

(d) Coat pile shaft with bitumen to allow slippage.

Bitumen compounds which can be sprayed or poured on clean piles are available to reduce negative skin friction. Coatings should be applied only to those portions of the pile anticipated to be within a zone of subsidence and the lower portion of the pile (at least ten times the diameter) should remain uncoated so that the full lower shaft and point resistance may be mobilized. Reductions of negative friction of 50% or greater have been measured for bituminous coatings on concrete and steel piling (see Reference 17, Reducing Negative Skin Friction with Bitumen Layers, by Claessen and Horvat, and Reference 18, Reduction of Negative Skin Friction on Steel Piles to Rock, by Bjerrum, et al.).

Section 4. PILE INSTALLATION AND LOAD TESTS

1. PILE INSTALLATION.

a. General Criteria. See Table 6.

b. Installation Techniques. Table 7 summarizes the more common supplementary procedures and appurtenances used in driven pile installations.

c. Pile Driving Hammers. Table 8 (Reference 6) summarizes the characteristics of the more common types of hammers in use in the U.S. Figure 5 shows principal operation of pile drivers (modified from Reference 6):

(1) Drop Hammer. Generally, it is only appropriate on small, relatively inaccessible jobs due to their slow rate of blows.

(2) Single Action Steam or Air Hammers. Blow rate is higher than drop hammer with maximum speeds generally ranging from about 35 to 60 blows per minute. Single acting hammers have an advantage over double acting hammers when driving piles in firm cohesive soils since the slower rate allows the soil and pile to relax before striking the next blow; thereby giving greater penetration per blow. In driving batter piles, single acting hammers can lose considerable energy due to the shortening fall and increases in friction.

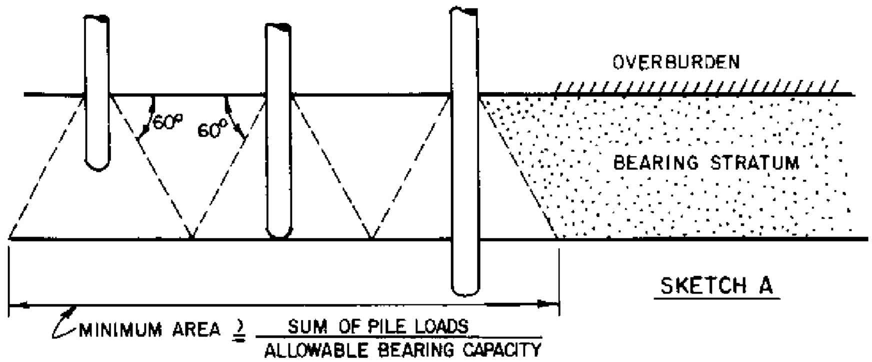
(3) Double Acting Steam or Air Hammers. They provide a blow rate nearly double that of the single acting hammers and lose less energy driving batter piles. They are generally best suited for driving piles in granular soils or in soft clays. The energy per blow delivered by a double-acting hammer decreases rapidly as its speed of operation drops below the rated speed.

(4) Diesel Hammers. They have a relatively low fuel consumption, operate without auxiliary equipment, and can operate at low temperatures and are more efficient for driving batter piles. Maximum blow rates are about 35 to 60 blows per minute for single acting and about 80 to 100 blows per minute for double acting. Diesel hammers operate best in medium to hard ground; in soft ground the resistance and resulting compression may be too low to ignite the fuel.

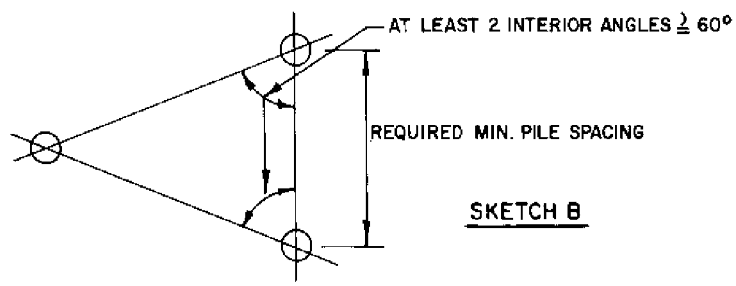
(5) Vibratory Hammers. They are best suited to wet soils and low displacement piles but occasionally have been used successfully in cohesive soils and with high displacement piles. They can also be effective in extracting piles. When conditions are suitable, vibratory hammers have several advantages over impact hammers including lower driving vibrations, reduced noise, greater speed of penetration and virtually complete elimination of pile damage. However, there is the possibility that the pile may not be efficiently advanced, obstructions generally can not be penetrated, and there is no generally accepted method of determining ultimate pile capacity based on the rate of penetration.

TABLE 6
General Criteria for Installation of Pile Foundations

GEOMETRIC REQUIREMENTS



SKETCH A



SKETCH B

ITEM	CRITERIA AND LIMITATIONS
GENERAL REQUIREMENTS MINIMUM SPACING (CENTER TO CENTER)	(1) PILES TO ROCK: TWICE THE AVERAGE PILE DIAMETER OR 1.75 TIMES THE DIAGONAL DIMENSION OF PILE CROSS SECTION, BUT NO LESS THAN 24". (2) ALL OTHER PILES: TWICE THE AVERAGE DIAMETER OF THE PILE OR 1.75 TIMES THE DIAGONAL DIMENSION OF PILE CROSS SECTION, BUT NO LESS THAN 30." IN ADDITION, THE MINIMUM SPACING SHALL BE LIMITED BY THE REQUIREMENT THAT THE PILE LOAD DISTRIBUTED INTO THE BEARING STRATUM SHALL NOT EXCEED THE NOMINAL BEARING CAPACITY OF THE STRATUM (TABLE 1, CHAPTER 4.). PILES OR PILE GROUPS SHALL BE ASSUMED TO TRANSFER THEIR LOADS TO THE UNDERLYING MATERIALS BY SPREADING THE LOAD UNIFORMLY AT AN ANGLE OF 60° WITH THE HORIZONTAL, STARTING AT A POLYGON CIRCUMSCRIBING THE PILES AT THE TOP OF THE BEARING STRATUM IN WHICH THEY ARE EMBEDDED. THE AREA CONSIDERED AS SUPPORTING THE LOAD SHALL NOT EXTEND BEYOND THE INTERSECTION OF THE 60° PLANES OF ADJACENT PILES OR PILE GROUPS. (SEE SKETCH A)
MINIMUM NUMBER OF PILES IN GROUP	PILE GROUPS SUPPORTING SUPERSTRUCTURE LOADS NORMALLY CONSIST OF AT LEAST 3 PILES (FOR ARRANGEMENT SEE SKETCH B), EXCEPT FOR INDIVIDUAL PILES SUPPORTING THE FLOOR SLAB OR IN CASES WHERE LATERAL TIES ARE PROVIDED.

TABLE 6 (continued)
General Criteria for Installation of Pile Foundations

Item	Criteria and Limitations
	Single pile supports may be used if the pile has a butt diameter of 12" or greater, if the upper soils are not of a weak nature, and if proper consideration is given to reinforcement of column and pile to accommodate potential eccentricities.
Embedment in pile cap.	Tops of piles shall extend at least 4" into the pile cap.
Pile length...........	No pile shall be shorter than 10 feet.
Tolerances in pile location and alinement	(1) Vertical piles shall not vary more than 2 percent from the plumb position.
	(2) No pile shall be driven more than 4" in horizontal dimension from its design location, unless the effect of this deviation is analyzed and found acceptable
	(3) Eccentricity of reaction of the pile group with respect to the load resultant shall not exceed a dimension that would produce overloads of more than 10 percent in any pile.
Driving Order.........	Pile groups shall be driven from the interior outward to preclude densification and excessively hard driving conditions on the interior.
Allowable loads:	
Allowable overload of piles................	(1) Up to 10 percent overload is permitted due to eccentricity of reaction of the pile group.
	(2) Overload due to wind is permitted if it does not exceed 33 percent of allowable capacity of pile under dead plus live loads.

TABLE 6 (continued)
General Criteria for Installation of Pile Foundations

Item	Criteria and Limitations
Lateral loads on vertical piles........	Maximum 1 ton per pile, if pile is embedded in soil for its entire length, except that no lateral load is permitted on vertical piles in very soft fine-grained soils or very loose coarse-grained soils. For piles with unsupported length or for larger horizontal loads, use batter piles or use analysis of Figure 10 to determine lateral load capacity of vertical piles.
Relative load capacity of piles in a group...	All bearing piles within a group shall be of the same type and be of equal load capacity.
Maximum allowable pile load..................	Shall be limited by both allowable stress in pile as given in Table 1 and supporting capacity of soil.
Static and dynamic pick-up loads.......	Induced flexural stresses incurred during pick-up and placement of long concrete piles shall not exceed the allowable bending stresses prescribed for that pile length.
Splices..............	Shall be able to transmit the resultant vertical and lateral forces adequately.
Load tests:	
Conditions requiring tests.................	Load tests to be performed for any of the following condition:
	(1) To verify or modify estimate of pile load capacity determined by other means.

TABLE 6 (continued)
General Criteria for Installation of Pile Foundations

Item	Criteria and Limitations
	(2) Where size of project and soil conditions indicate a significant savings is possible.
	(3) Where unique or unfamiliar types are to be used.
	(4) Where bearing stratum is underlain by a more compressible or questionable stratum.
Number of load tests..	A minimum of 3 test piles shall be driven per installation with uniform subsoil conditions. Two of these piles shall be test loaded, but no less than 1 load test for each 15,000 square feet of building area.
<u>Supervision</u>:	
Inspection............	All pile driving projects shall have on the site inspection by a person who has experience in such work, preferably a Registered Professional Engineer.
Records...............	Records shall be kept for the driving of each pile. The record shall include: date of driving, type, size, length, deviation from design location and alignment, pile hammer used, hammer speed, type and condition of cushion, and blows per foot for each foot of penetration for the full length of the pile, blows per inch for the final 6 inches of driving, except where an abrupt high increase in resistance is encountered, the final counts may be reduced to penetration for the last 5 blows.
General items to be checked...............	Material, quality of the pile straightness, application of preservatives, radiographic inspection of marine piling welds. For light weight mandrel driven shell piles, check interior for damage prior to concreting, check driving equipment for operational capabilities.

TABLE 7

Supplementary Procedures and Appurtenances Used in Pile Driving

Method	Equipment and procedure utilized	Applicability
Means of reducing driving resistance above bearing stratum:		
Temporary casing	Open end pipe casing driven and cleaned out. May be pulled later.	a. To drive through minor obstructions. b. To minimize displacement. c. To prevent caving or squeezing of holes. d. To permit concreting of pile before excavation to subgrade of foundation.
Precoring	By continuous flight auger or churn drill, a hole is formed into which the pile is lowered. Pile is then driven to bearing below the cored hole.	a. To drive through thick stratum of stiff to hard clay. b. To avoid displacement and heave of surrounding soil. c. To avoid injury to timber and thin shell pipes. d. To eliminate driving resistance in strata unsuitable for bearing.
Spudding	Heavy structural sections or closed end pipes are alternately raised and dropped to form a hole into which pile is lowered. Pile is then driven to bearing below the spudded hole.	a. To drive past individual obstruction b. To drive through strata of fill with large boulders or rock fragments.
Jetting	Water, air, or mixture of both forced through pipe at high pressures and velocity, jets are sometimes built into piles.	a. Used to facilitate penetration, should not be permitted in fine grained, poorly draining soils where frictional support may be permanently destroyed. Piles should be driven to final embedment after jetting.
Means of increasing driving resistance in bearing stratum:		
Upside down piles	Tapered piles, specifically timber, driven with large butt downward.	a. For end bearing timber piles, where it is necessary to minimize penetration into bearing stratum. b. To avoid driving through to incompressible but unsuitable bearing material.
Lagging	Short timber or steel sections connected by bolting or welding to timber or steel pipes.	a. To increase frictional resistance along sides of pile. b. To increase end bearing resistance when mounted near tip.
Means of overcoming obstructions:		
Shoes and reinforced tips.	Metal reinforcing, such as bands and shoes for all types of piles.	a. To provide protection against damage of tip. b. To provide additional cutting power.
Explosives	Drill and blast ahead of pile tip ..	a. To remove obstructions to open end piles under very severe conditions.
Preexcavation	Hand or machine excavation	a. Used for removal of obstruction close to ground surface.
Special equipment for advancing piles:		
Jacking	Hydraulic or mechanical screw jacks are used to advance pile. Pile is built up in short, convenient lengths.	a. To be used instead of pile hammer where access is difficult. b. To eliminate vibrations.
Vibration	High amplitude vibrators	a. Advantageous for driving in waterlogged sands and gravel. b. Advantageous for driving sheetpiling.
Follower	Temporary filler section between hammer and pile top, preferably of same material as pile.	a. To drive pile top to elevation below reach of hammer or below water.

TABLE 8
Impact and Vibratory Pile-Driver Data

1. IMPACT PILE HAMMER

Rated[**] Energy Kip - ft.	Make of Hammer[*]	Model No.	Types[*]	Blows per min	Stroke at Rated Energy	Striking Parts Kips	Weight Total Weight Kips
180.0	Vulcan	060	S-A	62	36	60.0	121.0
130.0	MKT	S-40	S-A	55	39	40.0	96.0
120.0	Vulcan	040	S-A	60	36	40.0	87.5
113.5	S-Vulcan	400C	Diff.	100	16.5	40.0	83.0
97.5	MKT	S-30	S-A	60	39	30.0	86.0
79.6	Kobe	K42	Dies.	52	98	9.2	22.0
60.0	Vulcan	020	S-A	60	36	20.0	39.0
60.0	MKT	S20	S-A	60	36	20.0	38.6
56.5	Kobe	K32	Dies.	52	98	7.0	15.4
50.2	S-Vulcan	200C	Diff.	98	15.5	20.0	39.0
48.7	Vulcan	016	S-A	60	36	16.2	30.2
48.7	Raymond	0000	S-A	46	39	15.0	23.0
44.5	Kobe	K22	Dies.	52	98	4.8	10.6
42.0	Vulcan	014	S-A	60	36	14.0	27.5
40.6	Raymond	000	S-A	50	39	12.5	21.0
39.8	Delmag	D-22	Dies.	52	N/A	4.8	10.0
37.5	MKT	S14	S-A	60	32	14.0	31.6
36.0	S-Vulcan	140C	Diff.	103	15.5	14.0	27.9
32.5	MKT	S10	S-A	55	39	10.0	22.2
32.5	Vulcan	010	S-A	50	39	10.0	18.7
32.5	Raymond	00	S-A	50	39	10.0	18.5
32.0	MKT	DE-40	Dies.	48	96	4.0	11.2
30.2	Vulcan	OR	S-A	50	39	9.3	16.7
26.3	Link-Belt	520	Dies.	82	43.2	5.0	12.5
26.0	MKT	C-8	D-A	81	20	8.0	18.7
26.0	Vulcan	08	S-A	50	39	8.0	16.7
26.0	MKT	S8	S-A	55	39	8.0	18.1
24.4	S-Vulcan	80C	Diff.	111	16.2	8.0	17.8
24.4	Vulcan	8M	Diff.	111	N/A	8.0	18.4
24.3	Vulcan	0	S-A	50	39	7.5	16.2
24.0	MKT	C-826	D-A	90	18	8.0	17.7
22.6	Delmag	D-12	Dies.	51	N/A	2.7	5.4
22.4	MKT	DE-30	Dies.	48	96	2.8	9.0
24.4	Kobe	K13	Dies.	52	98	2.8	6.4
19.8	Union	K13	D-A	111	24	3.0	14.5
19.8	MKT	11B3	D-A	95	19	5.0	14.5
19.5	Vulcan	06	S-A	60	36	6.5	11.2
19.2	S-Vulcan	65C	Diff.	117	15.5	6.5	14.8
18.2	Link-Belt	440	Dies.	88	36.9	4.0	10.3

TABLE 8 (continued)
Impact and Vibratory Pile-Driver Data

Rated[**] Energy Kip - ft.	Make of Hammer[*]	Model No.	Types[*]	Blows per min	Stroke at Rated Energy	Weight Striking Parts Kips	Total Weight Kips
16.2	MKT	S5	5-A	60	39	5.0	12.3
16.0	MKT	DE-20	Dies.	46	96	2.0	6.3
16.0	MKT	CS	Comp.	110	18	5.0	11.8
15.1	S-Vulcan	50C	Diff.	120	15.5	5.0	11.7
15.1	Vulcan	5M	Diff.	120	15.5	5.0	12.9
15.0	Vulcan	1	5-A	60	36	5.0	10.1
15.0	Link-Belt	312	Dies.	100	30.9	3.8	10.3
13.1	MKT	10B3	D-A	105	19	3.0	10.6
12.7	Union	1	D-A	125	21	1.6	10.0
9.0	Delmag	D5	Dies.	51	N/A	1.1	2.4
9.0	MKT	C-3	D-A	130	16	3.0	8.5
9.0	MKT	S3	5-A	65	36	3.0	8.8
8.8	MKT	DE-10	Dies.	48	96	11.0	3.5
8.7	MIT	9B3	D-A	145	17	1.6	7.0
8.2	Union	1.5A	D-A	135	18	1.5	9.2
8.1	Link-Belt	180	Dies.	92	37.6	1.7	4.5
7.2	Vulcan	2	5-A	70	29.7	3.0	7.1
7.2	S-Vulcan	30C	Diff.	133	12.5	3.0	7.0
7.2	Vulcan	3M	Diff.	133	N/A	3.0	8.4
6.5	Link-Belt	105	Dies.	94	35.2	1.4	3.8
4.9	Vulcan	DGH900	Diff.	238	10	.9	5.0
3.6	Union	3	D-A	160	14	.7	4.7
3.6	MKT	7	D-A	225	9.5	.8	5.0
.4	Union	6	D-A	340	7	.1	.9
.4	Vulcan	DGH100A	Diff.	303	6	.1	.8
.4	MKT	3	D-A	400	5.7	.06	.7
.3	Union	7A	D-A	400	6	.08	.5

[*] Codes

MKT - McKiernan-Terry D-A - Double-Acting
S-Vulcan - Super-Vulcan Diff. - Differential
S-A - Single-Acting Dies. - Diesel
Comp. - Compound

[**]In calculations of pile capacities by dynamic formula, effective energy delivered by hammer should be used. Hammer energy is affected by pressures used to operate the hammer, stroke rate, etc. Double-acting, differential, and diesel hammers may operate at less than rated energies; double-acting hammers deliver significantly less than rated energy when operated at less than rated speed. Consult manufacturers.

TABLE 8 (continued)
Impact and Vibratory Pile-Driver Data

2. VIBRATORY DRIVERS

Make	Model	Total Weight Kips	Available HP	Frequency Range cps	Force Kips[***], Frequency cps
Foster	2-17	6.2	34	18-21	
(France)	2-35	9.1	70	14-19	62/19
	2-50	11.2	100	11-17	101/17
Menck	MVB22-30	4.8	50		48/
(Germany)	MVB65-30	2.0	7.5		14/
	MVB44-30	8.6	100		97/
Muller	MS-26	9.6	72		
(Germany)	MS-26D	16.1	145		
Uraga	VHD-1	8.4	40	16-20	43/20
(Japan)	VHD-2	11.9	80	16-20	86/20
	VHD-3	15.4	120	16-20	129/20
Bodine (USA)	B	22	1000	0-150	63/100 - 175/100
(Russia)	BT-5	2.9	37	42	48/42
	VPP-2	4.9	54	25	49/25
	100	4.0	37	13	44/13
	VP	11.0	80	6.7	35/7
	VP-4	25.9	208		198/

[***] Forces given are present maximums. These can usually be raised or lowered by changing weights in the oscillator.

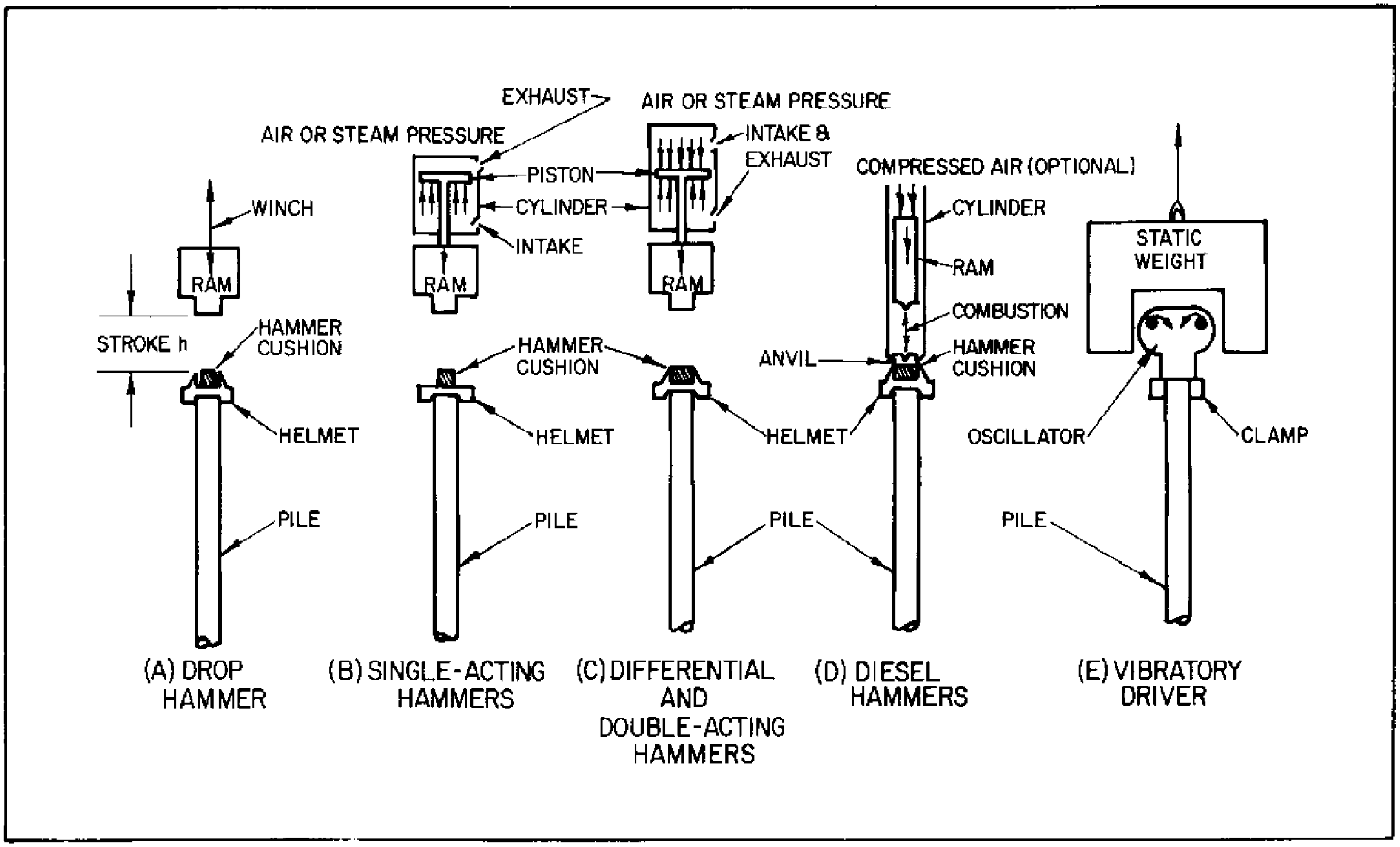


FIGURE 5
Principles of Operation of Pile Drivers

d. Inspection Guidelines. See Table 6 for general guidance and Reference 19, Inspectors' Manual for Pile Foundations, by the Deep Foundation Institute.

(1) Driven Piles. The inspector should normally assess the performance of the driving equipment, record the driving resistances, particularly the final set (net penetration per blow), record the driven depth and tip elevation, and continually observe the pile for evidence of damage or erratic driving. The criteria for termination of pile driving is normally a penetration resistance criteria or a required depth of penetration. Normally, a set criteria would be used for end bearing piles or piles where soil freeze is not a major factor while penetration criteria would be more appropriate for friction piles, piles into clay, and/or when soil freeze is a major factor.

(a) Timber Piles. (Reference 20, AWPI Technical Guidelines for Pressure-Treated Wood, Timber Piling, and ASTM Standard D25, Round Timber Piles.) Site Engineer/Inspector should check the following items:

- Overstressing at the top of pile, usually visible brooming.
- Properly fitted driving cap.
- Straightness.
- Sound wood free of decay and insect attack.
- Pressure treatment.
- Low frequency of knots.

(b) Concrete Piles. (Reference 21, Recommendations for Design, Manufacture, and Installation of Concrete Piles, by the American Concrete Institute.) Site Engineer/Inspector should check the following items:

- That pile length, geometry, thickness, and straightness conforms to specifications.
- Note extent, amount, and location of spilling or cracking in the pile during driving and pick up, and set.
- Thickness and type of cushion - should comply with specification.

(c) Steel Piles. Site Engineer/Inspector should check the following items:

- Compliance with applicable codes and specifications.
- Structural damage to pile due to over-driving/ overstressing.
- Pile orientation conforms to the plans.

(2) Drilled Piers. Minimum requirements for proper inspection of drilled shaft construction are as follows:

(a) For Dry or Casing Method of Construction;

- A qualified inspector should record the material types being removed from the hole as excavation proceeds.
- When the bearing soil has been encountered and identified and/or the designated tip elevation has been reached, the shaft walls and base should be observed for anomalies, unexpected soft soil conditions, obstructions or caving.
- Concrete placed freefall should not be allowed to hit the sidewalls of the excavation.
- Structural stability of the rebar cage should be maintained during the concrete pour to prevent buckling.
- The volume of concrete should be checked to ensure voids did not result during extraction of the casing.
- Concrete must be tremied into place with an adequate head to displace water or slurry if groundwater has entered the bore hole.
- Pulling casing with insufficient concrete inside should be restricted.
- Bottom of hole should be cleaned.

(b) For Slurry Displacement Method of Construction.

- A check on the concrete volume and recording the material types and depth of shaft apply the same as above.
- The tremie pipe should be watertight and should be fitted with some form of valve at the lower end.

(3) Caissons on Rock. Inspection of caisson bottom is usually accomplished by either:

(a) Probing with a 2-1/2" diameter probe hole to a minimum of 8 feet or 1.5 times the caisson shaft diameter (whichever is larger).

(b) Visual inspection by a qualified geologist at caisson bottom with proper safety precautions or from the surface utilizing a borehole camera. The purpose of the inspection is to determine the extent of seams, cavities and fractures. The allowable cumulative seam thickness within the probe depth varies depending on performance criteria. Values as low as 1/4" of cumulative thickness can be specified for the top 1/2 diameter.

e. Installation Guidelines.

(1) Driven Piles.

(a) For pile groups, drive interior piles first to avoid hard driving conditions, overstressing, and to minimize heave.

(b) Make sure pile driving caps and/or cushions are appropriate.

(c) Check for compression bands around the top of concrete and timber piles to avoid overstressing.

(d) Cheek for proper alignment of the driving head.

(e) If the pile suddenly changes directions or a substantially reduced driving resistance is noted, the pile is probably broken.

Table 9 summarizes some of the common installation problems and recommended procedures. Table 10 (Reference 22, Drilled Shafts: Design and Construction Guideline Manual, Vol 1: Construction Procedures and Design for Axial Load, by Reese and Wright) summarizes some of the more common installation problems and procedures for drilled piers.

(2) Performance Tolerance. It is normal practice to tailor the specifications to particular site conditions and to structural performance criteria. In many applications the following criteria may apply:

(a) Allowable Deviation from Specified Location. In the absence of another over-riding project specification criteria, use 4 inches. Consider the technical feasibility of increasing to more than 4 inches for caps with 4 piles or less.

(b) Allowable out-of-vertical. In the absence of the overriding project specification criteria, use 2% provided that the allowable deviation is not exceeded. Values of 4%, 2% and 1/4 inch out of plumb have been used.

(c) Allowable Heave Before Redriving. Require redriving of piles if heave exceeds 0.01 feet for essentially friction piles, or any detectable heave if piles are known to be essentially end-bearing.

(d) Minimum Distance of Pile Being Driven from Fresh Concrete. In the absence of over-riding project specification criteria, use 15 feet. Values of 10 feet to 50 feet have been used in practice.

TABLE [illegible]

Treatment of Field Problems Encountered during Pile Driving

Description of problem	Procedures to be applied
Category:	
Obstructions: Old foundations, boulders, rubble fill, cemented lenses, and similar obstacles to driving.	Excavate or break up shallow obstruction if practical. For deeper obstructions use spudding, jetting, or temporary casings, or use drive shoes and reinforced tips where pile is strong enough to be driven through obstructions.
General problems:	
Vibration in Driving: May compact loose granular materials causing settlement of existing structures near piles. Effect most pronounced in driving displacement piles.	Select pile type with minimum displacement, and/or precore or jet with temporary casing or substitute jacking for pile driving.
Damage to Thin Shells: Driven shells may have been crimped, buckled, or torn, or be leaking at joints as the results of driving difficulties or presence of obstructions.	Each pile is inspected with light beam. If diameter at any location varies more than 15% from original diameter or if other damage to shell cannot be repaired, pile is abandoned, filled with sand and a replacement is driven. Concrete shall be placed in dry shell only.
Inappropriate Use of Pile Driving Formula: Piles driven to a penetration determined solely by driving resistance may be bearing in a compressible stratum. This may occur in thick strata of silty fine sand, varved silts and clays, or medium stiff cohesive soils.	Unsuitable bearing strata should be determined by exploration program. Piles should not be permitted to stop in these strata, regardless of driving resistance. For bearing in stiff and brittle cohesive soils and in soft rock, load tests are particularly important.
Difficulties at pile tip:	
Fracturing of Bearing Materials: Fracturing of material immediately below tips of piles driven to required resistance as a result of driving adjacent piles. Brittle weathered rock, clay-shale, shale, siltstone, and sandstone are vulnerable materials. Swelling of stiff fissured clays or shales at pile tip may complicate this problem.	For piles bearing in these materials specify driving resistance test on selected piles after completion of driving adjacent piles. If damage to the bearing stratum is evidenced, require redriving until specified resistance is met.
Steeply Sloping Rock Surface: Tips of high capacity end bearing piles may slide or move laterally on a steeply sloping surface of sound hard rock which has little or no overlying weathered material.	Provide special shoes or pointed tips or use open end pipe pile socketed into sound rock.
Loss of Ground: May occur during installation of open end pipe piles. Materials vulnerable to piping, particularly fine sands or silts, may flow into pipe under the influence of an outside differential head, causing settlement in surrounding areas or loss of ground beneath tips of adjacent piles.	Avoid cleaning in advance of pile cutting edge, and/or retain sufficient material within pipe to prevent inflow of soil from below.
Movement of piles subsequent to driving:	
Heave: Completed piles rise vertically as the result of driving adjacent piles. Particularly common for displacement piles in soft clays and medium compact granular soils. Heave becomes serious in soft clays when volume displaced by piles exceeds 2½% of volume of soil enclosed within the limits of the pile foundation.	For piles of solid cross sections (timber, steel, precast concrete), survey top elevations during driving of adjacent piles to determine possible heave. For piles that have risen more than 0.01 ft, redrive to at least the former tip elevation, and beyond that as necessary to reach required driving resistance. Heave is minimized by driving temporary open-end casing, precoring, or jetting so that total volume displaced by pile driving is less than 2 or 3% of total volume enclosed within limits of pile foundation.
Lateral Movement of Piles: Completed piles move horizontally as the result of driving adjacent piles.	Survey horizontal position of completed piles during the driving of adjacent piles. Movement is controlled by procedures used to minimize heave.

TABLE 10
Drilled Piers: Construction Problems

Problem	Solution
Pouring concrete through water	Removal of water by hailing or use of tremie
Segregation of concrete during placing	If free-fall is employed, exercising care to see that concrete falls to final location without striking anything, or use of tremie
Restricted flow of concrete through or around rebar cage	Designing of rebar cage with adequate spacing for normal concrete (all clear spaces at least three times the size of largest aggregate) or use of special mix with small-sized coarse aggregate
Torsional buckling of rebar cage during concrete placement with casing method	Strengthening rebar cage by use of circumferential bands welded to lower portion of cage, use of concrete with improved flow characteristics, use of retarder in concrete allowing casing to be pulled very slowly
Pulling casing with insufficient concrete inside	Always having casing extending above ground surface and always having casing filled with a sufficient head of concrete with good flow characteristics before casing is pulled
Weak soil or undetected cavity beneath base of foundation	Requiring exploration to a depth of a few diameters below the bottom of the excavation
Deformation or collapse of soil	Such problems are readily detected by even the minimums of inspection

2. PILE LOAD TEST.

a. General. The results of pile load tests are the most reliable means of evaluating the load capacity of a deep foundation. Load tests can be performed during the design phase as a design tool and/or during construction to verify design loads. Pile load tests should be considered for large and/or critical projects, for pile types and soil conditions for which there is limited previous local experience, when proposed design loads exceed those normally used, and for other design/site conditions such as the need to use lower than specified factor of safety in the design.

The types of pile load tests normally performed include:

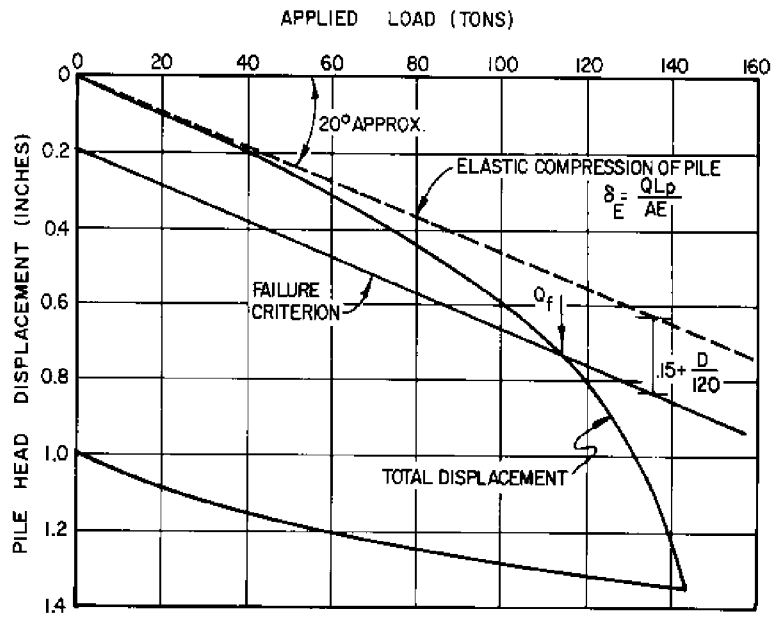
(1) Standard Loading Procedures or Slow Maintained-Load Test Method. For procedure, refer to ASTM Standard D3689, Individual Piles under Static Axial Tensile Load. It is the most common load test currently used. It is a long duration test (typically 70 hours or longer) loaded to 200 percent of the design load, or to failure. To determine curve of plastic deformation, the test procedure should be altered to include at least three unload-reload cycles. This procedure is described in ASTM Standard D1143, Pile Under Axial Compressive Load.

(2) Quick Maintained-Load Test Method. For procedure, refer to ASTM Standard D1143. This is a short duration test, typically 1 to 4 hours, generally loaded to 300 percent of the design load or failure. It is suitable for design load test and can be effectively used for load proof testing during construction.

(3) Constant Rate of Penetration (or Uplift) Test Method. A displacement-controlled method. For procedure, refer to ASTM Standard D1143 or ASTM Standard D3689. It is a short duration test, typically 2 to 3 hours, and may require special loading equipment as described in Reference 23, A Device for the Constant Rate of Penetration Test for Piles by Garneau and Samson. This method is recommended for testing piles in cohesive soils and for all tests where only the ultimate capacity is to be measured. The method can provide information regarding behavior of friction piles and is well suited for load tests during design.

b. Interpretation of Results. There are numerous procedures for interpretation of pile load test results including those specified by local building codes. A deflection criteria is normally used to define failure. In the absence of an over-riding project specification criteria, use 3/4 inch net settlement at twice the design load. Values of 1/4 and 1 inch at twice the design load and 1/4 inch at three times the design load have been used. Figure 6 presents a procedure for determining the failure load based on a permanent set of 0.15 + D/120 inches (where D is the pile diameter in inches). This procedure can be used for either of the three test methods presented above.

Where negative skin friction (downdrag) may act on the pile, only load carried by the pile below the compressible zone should be considered. This may be determined by minimizing shaft resistance during the load test (e.g., predrilling oversized hole, case and clean, using bentonite slurry, etc.) or by measuring movement of tip directly by extension rods attached to the pile tip and analyzing test results in accordance with Figure 7.



TYPICAL TEST PLOT

1. Calculate elastic compression of pile (δ_E) when considered as a free column by:

   $\delta_E = \frac{QL_p}{AE}$

   Q = test load, lbs
   L_p = pile length, in. (for end-bearing pile)
   A = cross-sectional area of pile material, sq in
   E = Young's Modulus for pile material, psi

2. Determine scales of plot such that slope of pile elastic compression line is approximately 20°.

3. Plot pile head total displacment vs. applied load.

4. Failure load is defined as that load which produces a displacement of the pile head equal to:

   $S_f = \delta_E + (.15 + \frac{D}{120})$

   S_f = displacement at failure, in.
   D = pile diameter, in.

5. Plot failure criterion as described in (4), represented as a straight line, parallel to line of pile elastic compression. Intersection of failure criterion with observed load deflection curve defines failure load, Q_f.

6. Where observed load displacement curve does not intersect failure criterion, the maximum test load should be taken as the failure load.

7. Apply factor of safety of at least 2.0 to failure load to determine allowable load.

FIGURE 6
Interpretation of Pile Load Test

c. Pullout Tests. Methods of determining failure load for tension load tests vary depending on the tolerable movement of the structure. In general, failure load is more easily defined than for compression load tests since available resistance generally decreases more distinctly after reaching failure. Failure load may be taken as that value at which upward movement suddenly increases disproportionately to load applied, i.e. the point of sharpest curvature on the load-displacement curve.

d. Lateral Load Tests. Lateral load tests are usually performed by jacking apart two adjacent pile and recording deflections of the piles for each load increment. See Reference 24, Model Study of Laterally Loaded Pile, by Davisson and Salley, for further guidance. In some applications testing of a pile group may be required.

e. Other Comments. A response of a driven pile in a load test can be greatly affected by the time elapsed between driving and testing. In most cases, a gain in pile bearing capacity is experienced with time and is governed by the rate of dissipation of excess pore water pressures generated by driving the pile throughout the surrounding soil mass. This is frequently termed "freezing." The time required for the soil to regain its maximum shear strength can range from a minimum of 3 to 30 days or longer. The actual required waiting period may be determined by redriving piles or from previous experience. Generally, however, early testing will result in an underestimate of the actual pile capacity especially for piles deriving their capacity from saturated cohesive soils.

Piles driven through saturated dense fine sands and silts may experience loss of driving resistance after periods of rest. When redriven after periods of rest the driving resistance (and bearing capacity) will be less compared to the initial driving resistance (and capacity). This phenomenon is commonly referred to as relaxation.

Section 5. DISTRIBUTION OF LOADS ON PILE GROUPS

1. VERTICAL PILE GROUPS.

a. Eccentric Vertical Loading. Distribution of design load on piles in groups is analyzed by routine procedures as follows:

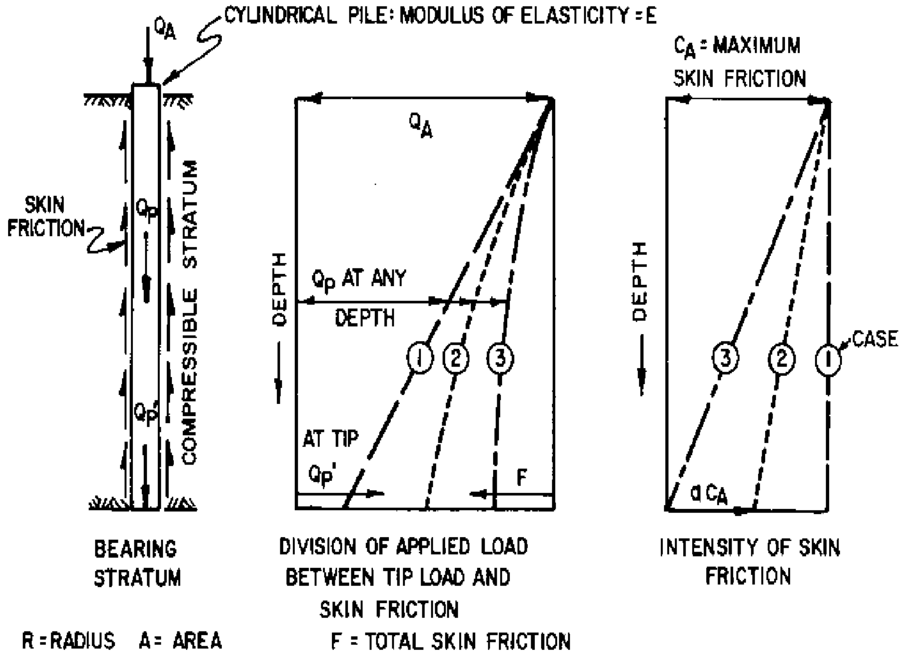
(1) For distribution of applied load eccentric about one or two axes, see Reference 6.

(2) Overload from eccentricity between applied load and center of gravity of pile group shall be permitted up to 10 percent of allowable working load when a safety factor of 2-1/2 to 3 is available for the working load.

(3) Overload from wind plus other temporary live loads up to 33 percent of the allowable working load is permitted, when a safety factor of 2-1/2 to 3 is available for the working load.

(4) Except in unusual circumstances, all bearing piles in a group shall be of the same type, and of equal load capacity.

1. IF SKIN FRICTION ACTING ON TEST PILE MAY BE REVERSED IN THE PROTOTYPE BY CONSOLIDATION OF MATERIALS ABOVE THE BEARING STRATUM, ANALYZE LOAD TEST TO DETERMINE RELATION OF LOAD VS SETTLEMENT FOR PILE TIP ALONE.
2. COMPUTE THEORETICAL ELASTIC SHORTENING ASSUMING SEVERAL POSSIBLE VARIATIONS OF SKIN FRICTION ON PILE AS SHOWN BELOW FOR A CYLINDRICAL PILE.
3. COMPARE THEORETICAL WITH OBSERVED ELASTIC SHORTENING AND DETERMINE PROBABLE VARIATION OF SKIN FRICTION ON PILE. USING THIS VARIATION OF SKIN FRICTION, COMPUTE LOAD AT TIP.



δ_E = ELASTIC SHORTENING OF PILE WITH LOAD Q_A AT BUTT AND Q_P' AT TIP.

CASE 1, SKIN FRICTION CONSTANT WITH DEPTH:

$$\delta_E = (Q_A - \pi R C_A L)\frac{L}{AE}$$

$$Q_P' = \frac{2AE\delta_E}{L} - Q_A$$

CASE 2, SKIN FRICTION DECREASING TO $a C_A$ AT TIP:

$$\delta_E = \left(Q_A - 2\pi R C_A L\left(\frac{a+2}{3(a+1)}\right)\right)\frac{L}{AE}$$

$$Q_P' = \frac{3AE\delta_E}{L}\left(\frac{a+1}{a+2}\right) - Q_A\left(\frac{2a+1}{a+2}\right)$$

CASE 3, SKIN FRICTION DECREASING TO ZERO AT TIP:

$$\delta_E = \left(Q_A - \frac{4\pi R C_A L}{3}\right)\frac{L}{AE}$$

$$Q_P' = \frac{3AE\delta_E}{2L} - \frac{Q_A}{2}$$

FIGURE 7
Load Test Analysis Where Downdrag Acts on Pile

2. GROUPS WITH VERTICAL AND BATTER PILES. Analyze distribution of pile loads according to criteria in Reference 25, Pile Foundations, by Chellis. The following limitations apply:

(1) Assume inclination of batter piles no flatter than 1 horizontal to 3 vertical unless special driving equipment is specified.

(2) When batter piles are included in a group, no allowance is made for possible resistance of vertical piles to horizontal forces.

(3) For analysis of loads on piles in relieving platforms, see Reference 26, American Civil Engineering Practice, Vol. 1, by Abbett.

(4) For analysis of batter pile anchorage for tower guys, see Figure 8.

Section 6. DEEP FOUNDATIONS ON ROCK

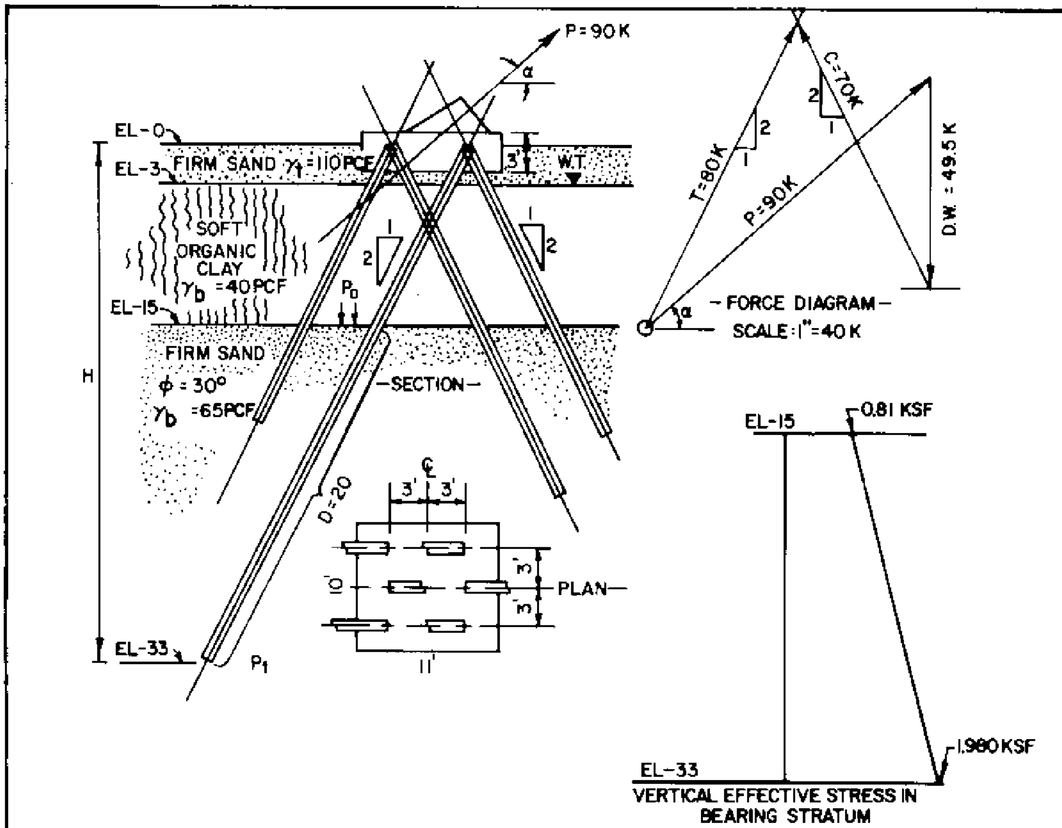
1. GENERAL. For ordinary structures, most rock formations provide an ideal foundation capable of supporting large loads with negligible settlement. Normally, the allowable loads on piles driven into rock are based on pile structural capacity while the allowable bearing pressures for footings/piers on rock are based on a nominal values of allowable bearing capacity (see Chapter 4).

There are however certain unfavorable rock conditions (e.g., cavernous limestone, see DM-7.1, Chapter 1) which can result in excessive settlement and/or failure. These potential hazards must be considered in the design and construction of foundations on rock.

2. PILES DRIVEN INTO ROCK. Piles driven into rock normally meet refusal at a nominal depth below the weathered zone and can be designed based on the structural capacity of the pile imposed by both the dynamic driving stresses and the static stresses. Highly weathered rocks such as decomposed granite or limestone and weakly cemented rocks such as soft clay-shales can be treated as soils.

The possibility of buckling below the mudline should be evaluated for high capacity pile driven through soft soils into bedrock (see Reference 27, The Design of Foundations for Buildings, by Johnson and Kavanaugh).

3. ALLOWABLE LOADS ON PIERS IN ROCK. Piers drilled through soil and a nominal depth into bedrock should be designed on the basis of an allowable bearing pressure given in Chapter 4 or other criteria (see Reference 28, Foundation Engineering, by Peck, et al.). Piers are normally drilled a nominal depth into the rock to ensure bearing entirely on rock and to extend the pier through the upper, more fractured zones of the rock. Increase in allowable bearing with embedment depth should be based on encountering more competent rock with depth.



FOR A SIX PILE GROUP -4 TENSION, 2 COMPRESSION ASSUME A 14" SQUARE STRAIGHT CONCRETE PILE, AS SHOWN, D=20' (D<20 B)

PILE CAP WEIGHT (D_W) = 10 X 11 X 3 X 0.15 KCF = 49.5 K

FIND F_s AGAINST Q_{ult} AND T_{ult}

DETERMINE PILE FORCES BY A FORCE DIAGRAM.

IGNORE RESISTANCE ABOVE FIRM SAND STRATUM.

BEARING STRATUM $\phi = 30^\circ$, $N_q = 21$

$$K_{HC} = 1.5,\ K_{HT} = 1.0,\ \delta = \frac{3}{4}(30^\circ) = 22.5^\circ \quad \text{(FROM FIGURE 1)}$$

$A_T = (\frac{14}{12})^2 = 1.36$ SF

PERIM. AREA/ℓf = $4 \times \frac{14}{12} = 4.7$ SF/ℓf

$$Q_{ult} = 1.98 \times 21 \times 1.36 + (1.5 \times (\frac{0.81+1.98}{2}) \times \tan 22.5 \times 4.7 \times 20)$$
$$= 56.54 + 81.47$$
$$= 138\text{ K}$$

COMPRESSION LOAD ON PILES FROM FORCE DIAGRAM IS 70 KIPS. LOAD PER PILE $70\text{K}/2 = 35\text{K}$

$$F_S = \frac{138\text{ K}}{35\text{K}} = 3.9 > 2,\ \text{REQ'D } F_s$$

$$T_{ult} = 1.0 \times (\frac{0.81+1.98}{2}) \times \tan 22.5 \times 4.7 \times 20$$
$$= 54.31\text{ K}$$

TENSION LOAD ON PILES FROM FORCE DIAGRAM IS 80 KIPS. LOAD PER PILE $\frac{80\text{K}}{4} = 20\text{ K}$

WEIGHT OF PILE = 0.204 Kp/ℓf X 37 = 7.6 K

$$F_S = \frac{54.3}{(20.0-7.6)} = 4.3 > 3\ \text{REQ'D } F_s$$

FIGURE 8
Example Problem - Batter Pile Group as Guy Anchorage

Rock-socketed drilled piers extending more than a nominal depth into rock derive capacity from both shaft resistance and end bearing. The proportion of the load transferred to end bearing depends on the relative stiffness of the rock to concrete and the shaft geometry. Generally, the proportion transferred to end bearing decreases for increasing depth of embedment and for increasing rock stiffness. This proportion increases with increased loading. Field tests indicate that the ultimate shaft resistance is developed with very little deformation (usually less than 0.25 inches) and that the peak resistance developed tends to remain constant with further movement. Based on load test data, the ultimate shaft resistance can be estimated approximately from:

S+r,= (2.3 to 3)(fw').1/2- (pier diameter >16 inches)
S+r,= (3 to 4)(fw').1/2- (pier diameter <16 inches)

where: S+r, = ultimate shaft resistance in force per shaft contact area

fw' = unconfined compressive strength of either the rock or the concrete, whichever is weakest.

See Reference 29, Shaft Resistance of Rock Socketed Drilled Piers, by Horvath and Kenney.

4. SETTLEMENT OF DEEP FOUNDATIONS IN ROCK. Settlement is normally negligible and need not be evaluated for foundations on rock designed for an appropriate allowable bearing pressure.

For very heavy or for extremely settlement sensitive structures, the settlement can be computed based on the solution for elastic settlement presented in Chapter 5 of DM-7.1. The choice of the elastic modulus, E, to use in the analysis should be based on the rock mass modulus which requires field investigation. For guidance see Reference 9 and Reference 30, Rock Mechanics in Engineering Practice, by Stagg and Zienkiewicz, eds. In cases where the seismic Young's modulus is known, the static modulus can be conservatively assumed to be 1/10th the seismic modulus.

Section 7. LATERAL LOAD CAPACITY

1. DESIGN CONCEPTS. A pile loaded by lateral thrust and/or moment at its top, resists the load by deflecting to mobilize the reaction of the surrounding soil. The magnitude and distribution of the resisting pressures are a function of the relative stiffness of pile and soil.

Design criteria is based on maximum combined stress in the piling, allowable deflection at the top or permissible bearing on the surrounding soil. Although 1/4-inch at the pile top is often used as a limit, the allowable lateral deflection should be based on the specific requirements of the structure.

2. DEFORMATION ANALYSIS - SINGLE PILE.

a. General. Methods are available (e.g., Reference 9 and Reference 31, Non-Dimensional Solutions for Laterally Loaded Piles, with Soil Modulus Assumed Proportional to Depth, by Reese and Matlock) for computing lateral pile load-deformation based on complex soil conditions and/or non-linear soil stress-strain relationships. The COM 622 computer program (Reference 32, Laterally Loaded Piles: Program Documentation, by Reese) has been documented and is widely used. Use of these methods should only be considered when the soil stress-strain properties are well understood.

Pile deformation and stress can be approximated through application of several simplified procedures based on idealized assumptions. The two basic approaches presented below depend on utilizing the concept of coefficient of lateral subgrade reaction. It is assumed that the lateral load does not exceed about 1/3 of the ultimate lateral load capacity.

b. Granular Soil and Normally to Slightly Overconsolidated Cohesive Soils. Pile deformation can be estimated assuming that the coefficient of subgrade reaction, K_h, increases linearly with depth in accordance with:

$$K_h = \frac{fz}{D}$$

where: K_h = coefficient of lateral subgrade reaction (tons/ft.^{3})

f = coefficient of variation of lateral subgrade reaction (tons/ft.^{3})
z = depth (feet)
D = width/diameter of loaded area (feet)

Guidance for selection of f is given in Figure 9 for fine-grained and coarse-grained soils.

c. Heavily Overconsolidated Cohesive Soils. For heavily overconsolidated hard cohesive soils, the coefficient of lateral subgrade reaction can be assumed to be constant with depth. The methods presented in Chapter 4 can be used for the analysis; K_h varies between 35c and 70c (units of force/length^3) where c is the undrained shear strength.

d. Loading Conditions. Three principal loading conditions are illustrated with the design procedures in Figure 10, using the influence diagrams of Figure 11, 12 and 13 (all from Reference 31). Loading may be limited by allowable deflection of pile top or by pile stresses.

Case I. Pile with flexible cap or hinged end condition. Thrust and moment are applied at the top, which is free to rotate. Obtain total deflections moment, and shear in the pile by algebraic sum of the effects of thrust and moment, given in Figure 11.

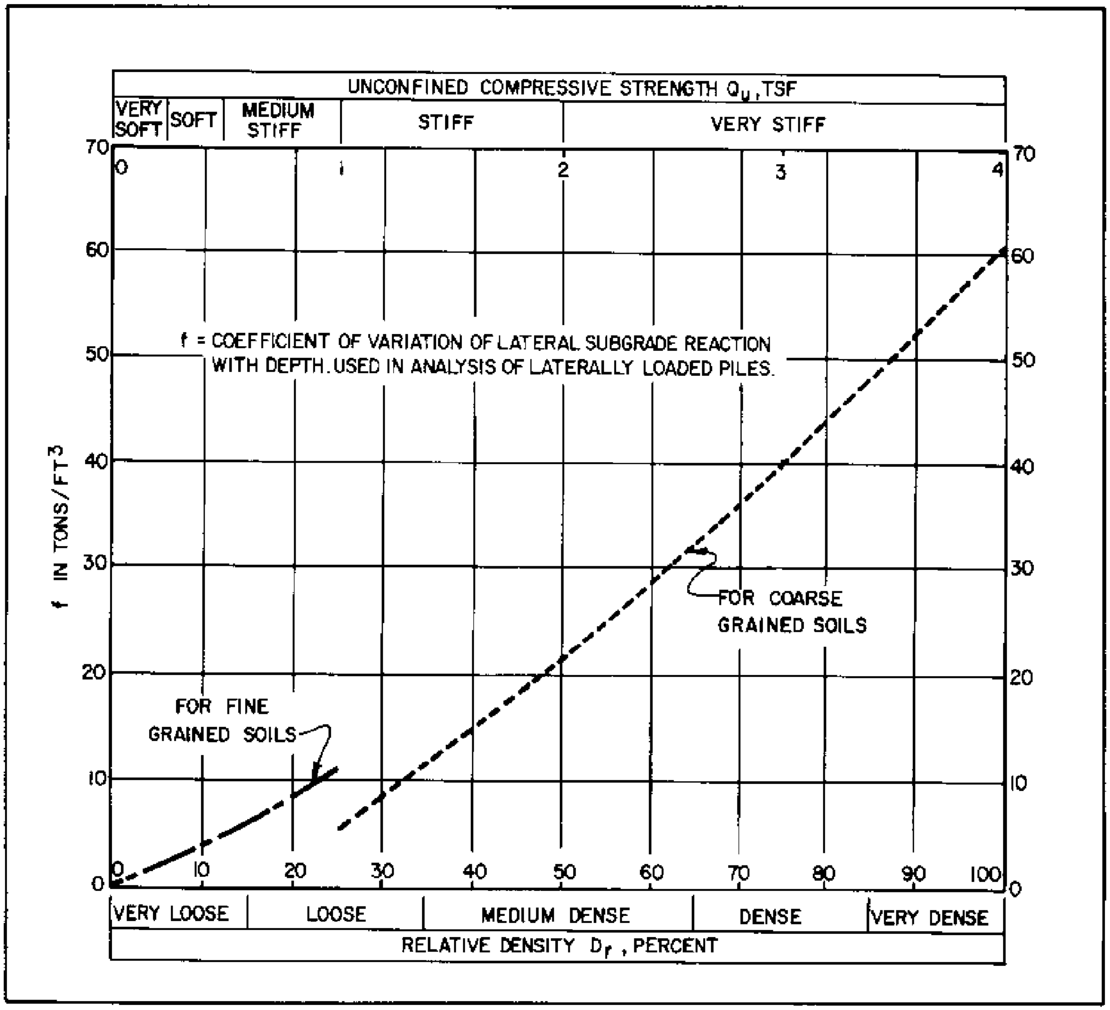


FIGURE 9
Coefficient of Variation of Subgrade Reaction

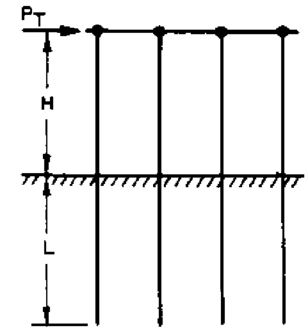
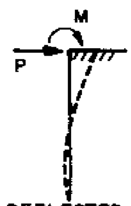
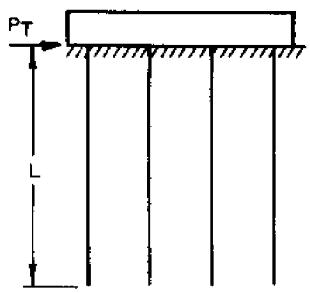
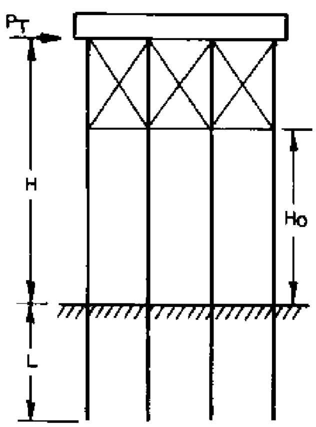
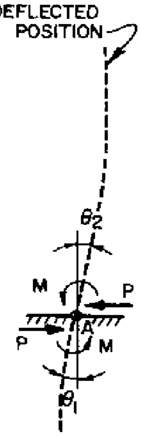
CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
CASE I. FLEXIBLE CAP, ELEVATED POSITION		
 n = NUMBER OF PILES	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$  DEFLECTED POSITION	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = (\frac{EI}{f})^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIGURE 11. 3. OBTAIN COEFFICIENTS F_δ, F_M, F_V AT DEPTHS DESIRED. 4. COMPUTE DEFLECTION, MOMENT AND SHEAR AT DESIRED DEPTHS USING FORMULAS OF FIGURE 11. NOTE: "f" VALUES FROM FIGURE 9 AND CONVERT TO LB/IN^3.
CASE II. PILES WITH RIGID CAP AT GROUND SURFACE		
		1. PROCEED AS IN STEP 1, CASE I. 2. COMPUTE DEFLECTION AND MOMENT AT DESIRED DEPTHS USING COEFFICIENTS F_δ, F_M AND FORMULAS OF FIGURE 12. 3. MAXIMUM SHEAR OCCURS AT TOP OF PILE AND EQUALS $P = \frac{P_T}{n}$ IN EACH PILE.
CASE III. RIGID CAP, ELEVATED POSITION		
		1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIGURE 13 AS FOLLOWS: $\theta_1 = F_\theta (\frac{PT^2}{EI}) + F_\theta (\frac{MT}{EI})$ 4. EQUATE $\theta_1 = \theta_2$ AND SOLVE FOR VALUE OF M. 5. KNOWING VALUES OF P AND M, SOLVE FOR DEFLECTION, SHEAR, AND MOMENT AS IN CASE I. NOTE: IF GROUND SURFACE AT PILE LOCATION IS INCLINED, LOAD P TAKEN BY EACH PILE IS PROPORTIONAL TO I/H_o^3.

FIGURE 10
Design Procedure for Laterally Loaded Piles

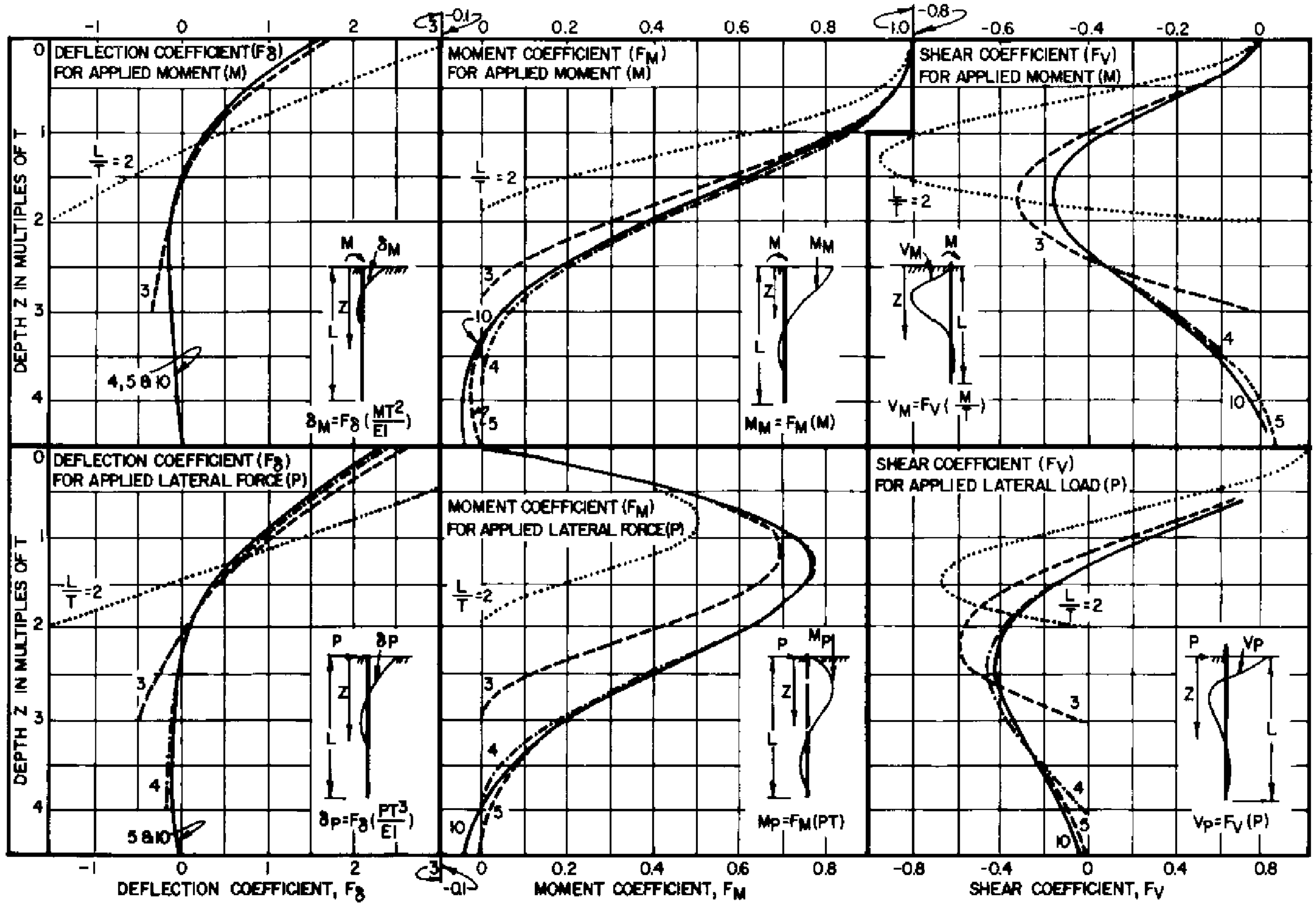


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

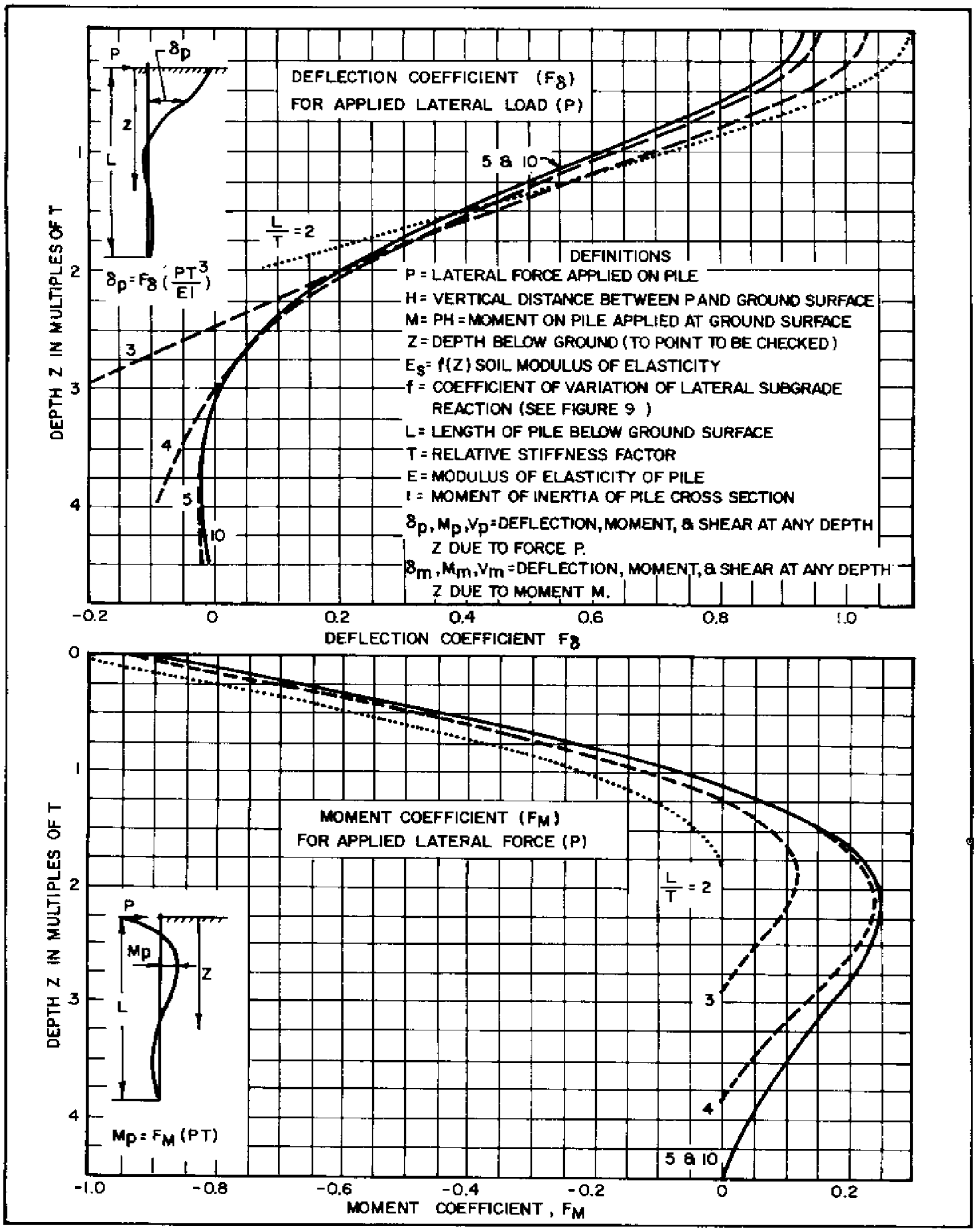
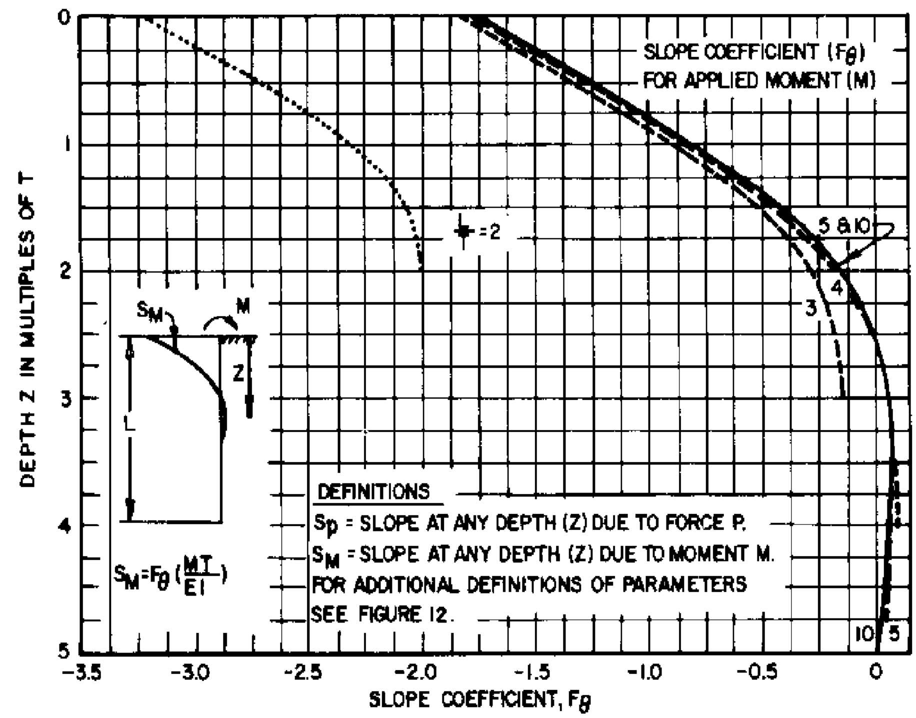


FIGURE 12
Influence Values for Laterally Loaded Pile
(Case II. Fixed Against Rotation at Ground Surface)



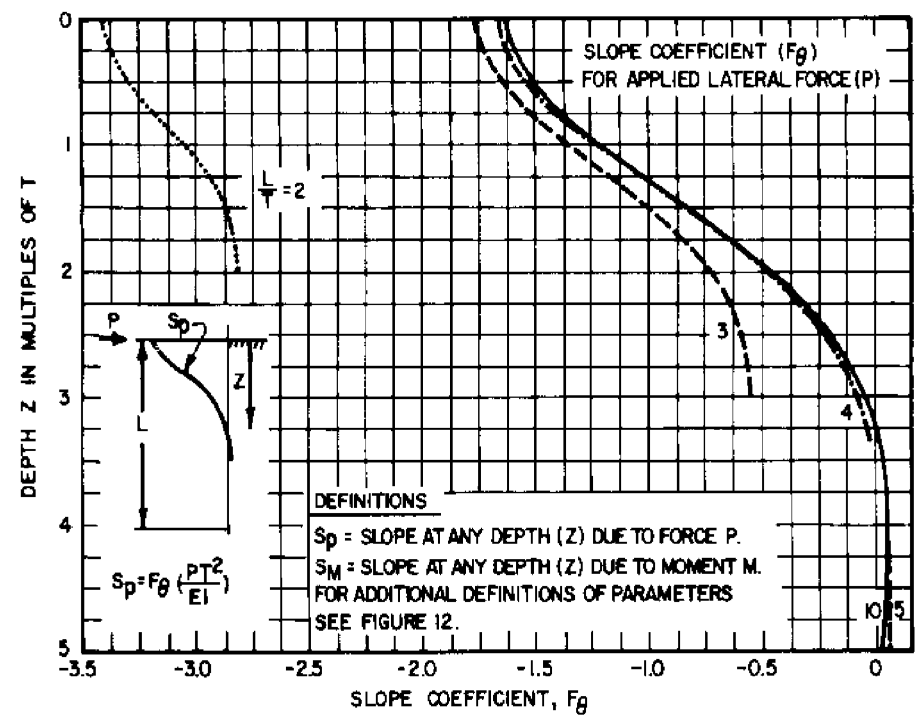


FIGURE 13
Slope Coefficient for Pile with Lateral Load or Moment

Case II. Pile with rigid cap fixed against rotation at ground surface. Thrust is applied at the top, which must maintain a vertical tangent. Obtain deflection and moment from influence values of Figure 12.

Case III. Pile with rigid cap above ground surface. Rotation of pile top depends on combined effect of superstructure and resistance below ground. Express rotation as a function of the influence values of Figure 13 and determine moment at pile top. Knowing thrust and moment applied at pile top, obtain total deflection, moment and shear in the pile by algebraic sum of the separate effects from Figure 11.

3. CYCLIC LOADS.

Lateral subgrade coefficient values decrease to about 25% the initial value due to cyclic loading for soft/loose soils and to about 50% the initial value for stiff/dense soils.

4. LONG-TERM LOADING. Long-term loading will increase pile deflection corresponding to a decrease in lateral subgrade reaction. To approximate this condition reduce the subgrade reaction values to 25% to 50% of their initial value for stiff clays, to 20% to 30% for soft clays, and to 80% to 90% for sands.

5. ULTIMATE LOAD CAPACITY - SINGLE PILES. A laterally loaded pile can fail by exceeding the strength of the surrounding soil or by exceeding the bending moment capacity of the pile resulting in a structural failure. Several methods are available for estimating the ultimate load capacity.

The method presented in Reference 33, <u>Lateral Resistance of Piles in Cohesive Soils</u>, by Broms, provides a simple procedure for estimating ultimate lateral capacity of piles.

6. GROUP ACTION. Group action should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the effective coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R (Reference 9) as follows:

Pile Spacing in Direction of Loading D = Pile Diameter	Subgrade Reaction Reduction Factor R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

REFERENCES

1. Teng, W.C., Foundation Design, Prentice Hall International, 1962

2. Skempton, A.W., The Bearing Capacity of Clays, Proceedings, Building Research Congress, London, 1951.

3. Tomlinson, M.F., The Adhesion of Piles Driven in Clay Soils, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, London, 1957.

4. Departments of the Army and Air Force, Soils and Geology, Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures), TM5/818-1/AFM88-3, Chapter 7, Washington, D.C. 1979.

5. Meyerhof, G.G., and Hanna, A.M., Ultimate Bearing Capacity of Foundations on Layered Soils Under Inclined Load Canadian Geotechnical Journal, Vol. 15, No. 4, 1978.

6. Vesic, A.S., Design of Pile Foundations, National Cooperative Highway Research Program Synthesis 42, Transportation Research Board, 1977.

7. Meyerhof, G.G., Bearing Capacity and Settlement of Pile Foundations, Journal of Geotechnical Engineering Division, ASCE, Vol. 102, No. GT3, 1976.

8. Baguelin, F., Jexequel, J.F., and Shields, D.H., The Pressuremeter and Foundation Engineering, TransTech Publications, 1978.

9. Canadian Geotechnical Society, Canadian Foundation Engineering Manual, Canadian Geotechnical Society, 1978.

10. Smith, E.A., Pile Driving by the Wave Equation, Transactions, American Society of Civil Engineers, Vol. 127, Part II, pp 1145-193, 1962.

11. Rausche, F., Moses, F., and Goble, G.G., Soil Resistance Predictions from Pile Dynamics, Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 98, No. SM9, 1972.

12. Meyerhof, G.G., Ultimate Bearing Capacity of Footings on Sand Layer Overlying Clay, Canadian Geotechnical Journal, Vol. 11, No. 2, 1974.

13. Whitaker, T., Experiments with Model Piles in Groups, Geotechnique, London, 1957.

14. Tomlinson, M.J., Pile Design and Construction Practice, Viewpoint Publications, Cement and Concrete Association, London, 1957.

15. Garlanger, J.E., Prediction of the Downdrag Load at Culter Circle Bridge, Symposium on Downdrag of Piles, Massachusetts Institute of Technology, 1973.

16. Fellenius, B.H., Downdrag on Piles Due to Negative Skin Friction, Canadian Geotechnical Journal, Vol. 9, No. 4, 1972.

17. Claessen, A.I.M. and Horvat, E., Reducing Negative Skin Friction with Bitumen Slip Layers, Journal of the Geotechnical Engineering Division, ASCE, Vol. 100, No. GT8, 1974.

18. Bjerrum, L., Johannessin, I.J., and Eide, O., Reduction of Negative Skin Friction on Steel Piles to Rock, Proceedings of the Seventh International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp 27-34, 1960.

19. Deep Foundation Institute, Inspectors' Manual for Deep Foundations, The Deep Foundations Institute, Springfield, NJ, 1978.

20. AWPI, Technical Guidelines for Pressure-Treated Wood, Timber Piling, P1 American Wood Preservers Institute, 1976.

21. ACI Committee 543, Recommendations for Design, Manufacture and Installation of Concrete Piles, ACI Manual of Concrete Practice, American Concrete Institute, Part 3, October, 1974.

22. Reese, L.C. and Wright,S.J., Drilled Shafts: Design and Construction, Guideline Manual, Vol. 1; Construction Procedures and Design for Axial Load, Federal Highway Authority, July, 1977.

23. Garneau, R., and Samson, L., A Device for the Constant Rate of Penetration Test for Piles, Canadian Geotechnical Journal, Vol. 11, No. 2, 1974.

24. Davisson, M.T. and Salley, J.R., Model Study of Laterally Loaded Pile, Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 96, No. SM5, 1970.

25. Chellis, R.D., Pile Foundations, McGraw Hill Book Company, 1961

26. Abbett, American Civil Engineering Practice, Vol. 1.

27. Johnson, S.M. and Kavanaugh, T.C., The Design of Foundations for Buildings, McGraw Hill Book Company, 1968.

28. Peck, R.B., Hanson, W.E., and Thornburn, T.H., Foundation Engineering, John Wiley & Sons, Inc., 1974.

29. Horvath, N.G. and Kenney, T.C., Shaft Resistance of Rock-Socketed Drilled Pier, ASCE, Preprint #3698, 1979.

30. Stagg, K.G. and Zienkiewiez, O.C., Eds., Rock Mechanics in Engineering Practice, John Wiley & Sons, Inc., 1968.

31. Reese, L.C. and Matlock, H., Non-Dimensional Solutions for Laterally Loaded Piles with Soil Modulus Assumed Proportional to Depth, Proceedings, Eighth Texas Conference on Soil Mechanics and Foundation Engineering, Austin, Texas, ASCE, 1956.

32. Reese, L.C., Laterally Loaded Piles: Program Documentation, Journal of Geotechnical Engineering Division, ASCE, Vol. 103, No. GT4, 1977.

33. Broms, B.B., Lateral Resistance of Piles in Cohesive Soils, Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 90, No. SM2 and SM3, 1964.

34. Naval Facilities Engineering Command

Design Manuals (DM)
- DM-25.06 General Criteria for Waterfront Construction
- DM-38.04 Pile Driving Equipment

Copies of Guide Specifications and Design Manuals may be obtained from the U.S. Naval Publications and Forms Center, 5801 Tabor Avenue, Philadelphia, PA 19120.

BIBLIOGRAPHY

American Petroleum Institute, Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms, API RP 2A, January 1971.

Casagrande, L., Comments on Conventional Design of Retaining Wall Structures, Journal of the Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 99, No. SM2, 1973.

Davisson, M.T., Inspection of Pile Driving Operations, Tech. Report M-22, Cold Regions Research Engineering Laboratories Corps of Engineers, U.S. Department of the Army, 1972.

Foster, C.R., Field Problems: Compaction, Foundation Engineering, Leonards, G.A., Editor, McGraw-Hill Book Co., Inc., New York, Chapter 12, 1952.

Fuller, F.N. and Hoy, H.E., Pile Load Tests Including Quick Load Test Method, Conventional Methods and Interpretations, Highway Research Record No. 333, HRB U.S. National Research Council, pp 74-86, 1970.

Hunt, H.W., Design and Installation of Pile Foundations, Associated Pile Fitting Corporation, Clifton, NJ, 1974.

McClelland, B., Design of Deep Penetration Piles for Ocean Structures, Terzaghi Lectures: 1963-1 72, ASCE, New York, pp 383-421, 1974.

Meyerhof, G.G., Bearing Capacity and Settlement of Pile Foundations, Journal of the Geotechnical Engineering Division, ASCE, Vol. 102, No. GT3, 1976.

National Bureau of Standards, Corrosion of Steel Pilings in Soils, Monographs 58 and 127, U.S. Department of Commerce, Washington, D.C.

Reese, L.C. and Welch, R.C., Lateral Loading of Deep Foundations in Stiff Clay, Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT7, 1975.

USBR, Earth Manual, Second Edition, U.S. Department of Interior, Bureau of Reclamation, U.S. Government Printing Office, Washington, D.C., 1974.

Vesic, A.S., Ultimate Loads and Settlements of Deep Foundations in Sand, Procs., Symposium on Bearing Capacity and Settlement of Foundations, Duke University, Durham, NC., 1967.

Winterkorn, H.F., and Fang, H.Y., Editors, Foundation Engineering Handbook, Van Nostrand Reinhold Co., New York, 1975.

Woodward, R.J., Gardner, W.S. and Greer, D.M., Drilled Pier Foundations, McGraw-Hill Book Co., Inc., New York, 1972.

APPENDIX A
Listing of Computer Programs

Subject	Program	Description	Availability
Shallow Foundations (Chapter 4)	QULT GESA Catalog No. E03-0001-00043	Bearing capacity analysis by Balla, Brinch Hansen, Meyerhof-Prandtl, Sokoluvski and Terzaghi Methods	Geotechnical Engineering Software Activity University of Colorado Boulder, CO 80309
Excavation, and Earth Pressures (Chapter 1) and (Chapter 3)	SOIL-STRUCT	Two dimensional finite element program to analyze tieback walls.	Stanford University
	SSTINCS-2DFE	Two dimensional finite element program to analyze tieback walls.	Virginia Polytechnic Institute and State University, Blacksburg, VA 24061
Deep Foundations (Chapter 5)	COM622 GESA Catalog No. E04-0003-00044	Program solves for deflection and bending moment in a laterally loaded pile based on theory of a beam on an elastic foundation using finite difference techniques. Soil properties are defined by a set of load-deflection curves.	GESA or University of Austin
	TTI	Program for analysis of pile driving by the Wave Equation; developed at Texas A & M University.	U.S. Department of Transportation FHWA R & D Implementation Div.
	WEAP GESA Catalog No. E04-0004-00046	Wave Equation analysis for pile driven by impact hammers, diesel hammers and air/steam hammers.	GESA

APPENDIX A
Listing of Computer Programs (continued)

Subject	Program	Description	Availability
Deep Foundations (Chapter 5)	WINIT GESA Catalog No. EO4-005-00047	Auxiliary program for WEAP.	GESA
	WEHAM GESA Catalog No. E04-006-00048	Hammer data for WEAP.	"
	WDATA GESA Catalog No. E04-007-00049	WEAP data generator.	"

GLOSSARY

Downdrag. Force induced on deep foundation resulting from downward movement of adjacent soil relative to foundation element. Also referred to as negative skin friction.

Homogenous Earth Dam. An earth dam whose embankment is formed of one soil type without a systematic zoning of fill materials.

Modulus of Subgrade Reaction. The ratio between the bearing pressure of a foundation and the corresponding settlement at a given point.

Nominal Bearing Pressures. Allowable bearing pressures for spread foundation on various soil types, derived from experience and general usage, which provide safety against shear failure or excessive settlement.

Optimum Moisture Content. The moisture content, determined from a laboratory compaction test, at which the maximum dry density of a soil is obtained using a specific effort of compaction.

Piping. The movement of soil particles as the result of unbalanced seepage forces produced by percolating water, leading to the development of boils or erosion channels.

Swell. Increase in soil volume, typically referring to volumetric expansion of particular soils due to changes in water content.

Zoned Earth Dam. An earth dam embankment zoned by the systematic distribution of soil types according to their strength and permeability characteristics, usually with a central impervious core and shells of coarser materials.

SYMBOLS

Symbol	Designation
A	Cross-sectional area.
A+p,	Anchor pull in tieback system for flexible wall.
B,b	Width in general, or narrow dimension of a foundation unit.
c+a,	Unit adhesion between soil and pile surface or surface of some other foundation material.
C+all,	Allowable cohesion that can be mobilized to resist shear stresses.
C+s,	Shape factor coefficient for computation of immediate settlement.
c	Cohesion intercept for Mohr's envelope of shear strength based on total stresses.
c'	Cohesion intercept for Mohr's envelope of shear strength based on effective stresses.
c+v,	Coefficient of consolidation.
D,d	Depth, diameter, or distance.
D+r,	Relative density.
D+5,, D+60, D+85,	Grain Size division of a Soil sample, percent of dry weight smaller than this grain size is indicated by subscript.
E	Modulus of elasticity of structural Material.
E+s,	Modulus of elasticity or "modulus of deformation" of soil.
e	Void ratio.
F+s,	Safety factor in stability or shear strength analysis.
f	Coefficient of variation of soil modulus of elasticity with depth for analysis of laterally loaded piles.
G	Specific gravity of solid particles in soil sample, or shear modulus of soil.
H,h	In general, height or thickness.
H+w,	Height of groundwater or of open water above a base level.
I	Influence value for vertical stress produced by superimposed load, equals ratio of stresses at a point in the foundation to intensity of applied load.
i	Gradient of groundwater pressures in underseepage analysis.
K+A,	Coefficient of active earth pressures.
K+H,	Ratio of horizontal to vertical earth pressures on side of pile or other foundation.
k+h,	Coefficient of lateral subgrade reaction.
K+p,	Coefficient of passive earth pressures.
K+b,	Modulus of subgrade reaction for bearing plate or foundation of width b.
K+v1,	Modulus of subgrade reaction for 1 ft square bearing plate at ground surface.
k	Coefficient of permeability.
ksf	Kips per sq ft pressure intensity.
ksi	Kips per sq in pressure intensity.

Symbol	Designation
L,1	Length in general or longest dimension of foundation unit.
N+c,, N+cs,, N+q,, N[gamma], N [gamma]q	Bearing capacity factors.
N+o,	Stability number for slope stability.
n	Porosity of soil sample.
n+c,	Effective porosity.
OMC	Optimum moisture content of compacted soil.
P+A,	Resultant active earth force.
P+AH,	Component of resultant active force in horizontal direction.
pcf	Density in pounds per cubic foot.
P+h,	Resultant horizontal earth force.
p+p,	Resultant passive earth force.
P+PH,	Component of resultant passive earth force in horizontal direction.
P+v,	Resultant vertical earth force.
P+w,	Resultant force of water pressure.
p	Intensity of applied load.
P+o,	Existing effective overburden pressure acting at a specific height in the soil profile.
P+c,	Preconsolidation pressure.
Q+all,	Allowable load capacity of deep foundation element.
Q+ult,	Ultimate load that causes shear failure of foundation unit.
q	Intensity of vertical load applied to foundation unit.
q+all,	Allowable bearing capacity of shallow foundation unit.
q+u,	Unconfined compressive strength of soil sample.
q+ult,	Ultimate bearing pressure that causes shear failure of foundation unit.
R,r	Radius of well or other right circular cylinder.
s	Shear strength of soil for a specific stress or condition in situ, used instead of strength parameters c and [theta].
T	Thickness of soil stratum, or relative stiffness factor of soil and pile in analysis of laterally loaded piles.
Z	Depth.
[gamma]+D,	Dry unit weight of soil.
[gamma]+E,	Effective unit weight of soil.
[gamma]+MAX,	Maximum dry unit weight of soil determined from moisture content dry unit weight curve; or, for cohesionless soil, by vibratory compaction.
[gamma]+MIN,	Minimum dry unit weight.
[gamma]+SUB,	Submerged (buoyant) unit weight of soil mass.
[gamma]+T,	Wet unit weight of soil above the groundwater table.
[gamma]+W,	Unit weight of water, varying from 62.4 pcf for fresh water to 64 pcf for sea water.
[RHO]	Magnitude of settlement for various conditions.
[phi]	Angle of internal friction or "angle of shearing resistance," obtained from Mohr's failure envelope for shear strength.
[Upsilon]	Poisson's Ratio.

INDEX

F

G

M

P

S

W

*U.S. GOVERNMENT PRINTING OFFICE : 1988 0 - 198-094