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DESIGN MANUAL

SOIL MECHANICS, FOUNDATIONS, AND EARTH STRUCTURES

NAVFAC DM-7
March 1971
(Including Change 1)

Metz Reference Room
University of Illinois
B106 NCEL
208 N. Romine Street
Urbana, Illinois 61801

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RECORD OF CHANGES

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ABSTRACT

Design guidance is presented for use by experienced engineers. The contents include soil classification; exploration and sampling; laboratory tests and test properties; field tests and measurements; distribution of stresses and pressures; analysis for settlement, stability, seepage, and drainage; compacted embankments, compaction procedures, and hydraulic fills; walls and retaining structures; spread foundations; deep foundations; pile foundations; pressures on buried structures; soil and rock stabilization; and frost, vibration, and seismic effects.

FOREWORD

This design manual for Soil Mechanics, Foundations and Earth Structures is one of a series that has been developed from an extensive reevaluation 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, other Government agencies, and private industry. This manual includes a modernization of the former criteria and the maximum use of national professional society, association and institute codes. Deviations from these criteria should not be made without the prior approval of the Naval Facilities Engineering Command Headquarters (NAVFAC HQ).

Design cannot remain static any more than can the naval functions it serves, or the technologies it uses. Accordingly, this edition of *Soil Mechanics, Foundations and Earth Structures*, NAVFAC DM-7 cancels and supersedes *Soil Mechanics, Foundations and Earth Structures*, NAVDOCKS DM-7, of February 1962 in its entirety, and all changes issued.

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PREFACE

This manual on soil mechanics, foundations, and earth structures covers the engineering application of soil mechanics to the design of all foundations and earth structures for naval shore facilities. These criteria, together with the definitive designs and guideline specifications of the Naval Facilities Engineering Command, constitute the Command's design guidance. These standards are based on functional requirements, engineering judgment, knowledge of materials and equipment, and the experience gained by the Naval Facilities Engineering Command and other commands and bureaus of the Navy in the design, construction, operation, and maintenance of naval shore facilities.

The design manual series presents criteria that shall be used in the design of facilities under the cognizance of the Naval Facilities Engineering Command. The direction and standards for procedures, methods, dimensions, materials, loads, and stresses will be included. Design manuals are not textbooks, but are for the use of experienced architects and engineers. Many criteria and standards appearing in technical texts issued by Government agencies, professional architectural and engineering groups, and trade and industry groups are suitable for, and have been made integral parts of, this series. The latest edition of each publication source shall be used.

Bibliographies of publications containing background information and additional reading on the various subjects are included in the manuals; this material however is not a part of the criteria, nor is a reading of these sources necessary for the use of the criteria presented in the manuals.

To avoid duplication and to facilitate future revisions, criteria are presented only once in this series as far as possible. Criteria having general applications appear in the basic manuals numbered DM-1 through DM-10 (numbers DM-11 through DM-20 were unassigned in the original issues). Manuals numbered DM-21 and above contain criteria that usually are applicable only to the specific facility class covered by each manual. When criteria for one facility also have an application in another facility class, the basic rule has been to present such criteria in the basic, or lowest numbered, manual and cite it by reference where required in later manuals.

The specific design manuals (DM-21 and above), with but three exceptions, list design criteria for specific facilities in the order of the category codes. The exceptions are:

(1) *Drydocking Facilities*, NAVFAC DM-29, which includes both .tegory codes 213 and 223.

(2) Criteria for facility class 800, Utilities and Ground Improvements, which have been included in the basic manuals on mechanical, electrical, and civil engineering.

(3) *Weight Handling Equipment and Service Craft*, NAVFAC DM-38, which includes the design criteria for these facilities under the cognizance of the Naval Facilities Engineering Command that are not classified as real property. These include weight and line handling equipment, dredges, yard craft, and piledriving equipment.

For the effective use of these criteria, the designer must have access to:

(1) The basic and specific design manuals applicable to the project. See list on page 7-viii.

(2) Published criteria sources.

(3) Applicable definitive designs, *Definitive Designs for Naval Shore Facilities*, NAVFAC P-272.

(4) Command guideline specifications.

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SYMBOLS

Symbol	*Designation*
A	Cross-sectional area.
A_c	Activity of fine grained soil.
A_p	Anchor pull in tieback system for flexible wall.
a_v	Coefficient of compressibility.
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 shear strength that can be mobilized to resist shear stresses.
CBR	California Bearing Ratio.
C_c	Coefficient of curvature of gradation curve or compression index for virgin consolidation.
CD	Consolidated-drained shear test.
C_r	Recompression index in reconsolidation.
C_s	Swelling index or shape factor coefficient for computation of immediate settlement.
CU	Consolidated-undrained shear test.
C_u	Coefficient of uniformity of grain size curve.
C_α	Coefficient of secondary compression.
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_r	Value of "true cohesion."
c_v	Coefficient of consolidation.
D, d	Depth, diameter, or distance.
D_d	Relative density.
D_e	ID of cutting edge of thin sampling tube.
D_w	OD of thin sampling tube.
D_{10}	Effective grain size of soil sample; 10 percent 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 ratio 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.
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.
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.
H_w	Height of ground water 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 ground water pressures in underseepage analysis.
K_A	Coefficient of active earth pressures.
K_f	Ratio of horizontal to vertical earth pressures on a vertical failure plane.
K_H	Ratio of horizontal to vertical earth pressures on side of pile or other foundation.

Symbol	*Designation*
K_p	Coefficient of passive earth pressures.
K_v	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 in general.
k_H	Coefficient of permeability in horizontal direction.
k_m	Mean coefficient of permeability of anistropic subsoil.
ksf	Kips per sq ft pressure intensity.
ksi	Kips per sq in. pressure intensity.
k_V	Coefficient of permeability in vertical direction.
L, l	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_c, N_{cs}, N_q, N_γ $N_{\gamma q}$	Bearing capacity factors.
N_o	Stability number for slope stability analysis.
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_h	Resultant horizontal earth force.
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.
p_c	Preconsolidation stress.
p_o	Existing effective overburden pressure acting at a specific height in the soil profile or on a soil sample.
Q_A	Total load applied to top of pile.
Q_D	Drag force exerted on sides of pile by consolidation of surrounding soil.
Q_P	Total load reaching pile tip.
Q_{all}	Allowable load capacity of pile.
Q_{ult}	Ultimate load capacity of pile.
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 ground water 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.

Symbol	Designation
SI	Shrinkage index.
SL	Shrinkage limit.
S_t	Sensitivity of soil, equals ratio of remolded to undisturbed shear strengths.
s	Shear strength of soil for a specific stress or condition in situ, used instead of strength parameters c and Φ.
T	Thickness of soil stratum, or relative stiffness factor of soil and pile in analysis of laterally loaded piles.
T_{max}	Maximum torque exerted in vane shear test of subsoils.
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 ground water pressures acting on a specific surface within the subsoils.
$\bar{U}$	Average degree of consolidation at any time.
u	Intensity of pore water pressures.
u_f	Increment of pore water pressures developed during shear of a triaxial test specimen.
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_D	Weight of pile driven by pile driver.
W_S	Weight of striking force of pile driver.
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.
γ_D	Dry unit weight of soil.
γ_{max}	Maximum dry unit weight of soil.
γ_{SAT}	Saturated unit weight of soil.
γ_{SUB}	Submerged (buoyant) unit weight of soil mass.
γ_T	Wet unit weight of soil above the ground water table.
γ_w	Unit weight of water, varying from 62.4 pcf for fresh water to 64 pcf for sea water.
ϵ	Unit strain in general.
ϵ_a	Axial strain in triaxial shear test.
Δe	Change in void ratio corresponding to a change in effective stress, Δp.
$\delta, \delta_v, \delta_c$	Magnitude of settlement for various conditions.
δ_E	Elastic shortening of pile.
Φ	Angle of internal friction or "angle of shearing resistance," obtained from Mohr's failure envelope for shear strength.
Φ_r	True angle of internal friction.
σ_1	Total major principal stress.
σ_3	Total minor principal stress.
$\bar{\sigma}_1$	Effective major principal stress.
$\bar{\sigma}_3$	Effective minor principal stress.
$\sigma_x, \sigma_y, \sigma_z$	Normal stresses in coordinate directions.

Symbol	Designation
τ	Intensity of shear stress.
τ_{max}	Intensity of maximum shear stresses.

ACKNOWLEDGMENTS

Figure or Table	*Acknowledgment*
Table 1-3	Terzaghi & Peck, *Soil Mechanics in Engineering Practice*, John Wiley & Sons, New York, N.Y.
Figure 3-7 (Upper Panel)	Bjerrum, L., and Simons, N..E., *Comparison of Shear Strength Characteristics of Normally Consolidated Clays*, Norwegian Geotechnical Institute, Publication No. 35, Oslo. (Modified)
Figure 4-1 (Upper Panel)	Holtz, W. E., and Gibbs, H. E., *Research on Determining the Density of Sands by Spoon Penetration Testing*, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, Butterworths Scientific Publications, London.
Figure 4-2 (Upper Right Panel)	Spangler, M. G., *Soil Engineering*, International Textbook Co., Scranton, Pa.
Table 4-5 (Illustrations for Pile Pulling Test)	Chellis, R. D., *Pile Foundations–Theory, Design and Practice*, McGraw-Hill Book Co., Inc., New York, N. Y.
Figure 5-3	Fadum, R. E., *Influence Values for Estimating Stresses in Elastic Foundations*, Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering.
Figure 5-4	Foster, C. R., and Ahlvin, P. G., *Stresses and Deflections Induced by Uniform Circular Load*, Highway Research Board Proceedings, 1954, Highway Research Board, Washington, D. C.
Figure 5-5	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, Butterwo' ' , Scientific Publications, London.
Figure 5-6	Gray, H., and Hooks, I. J., *Charts Facilitate Determination of Stresses Under Loaded Areas*, Civil Engineering, June 1948, American Society of Civil Engineers, New York, N.Y.
Figure 5-8	Jimenez Salas, J. A., *Soil Pressure Computations: A Modification of Newmark's Method*, Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering.
Figure 5-9	Mehta, M. R., and Veletsos, A. S., *Stresses and Displacement in Layered Systems*, Structural Research Series No. 178, University of Illinois, Urbana, Ill.
Figure 6-6	*Procedures for Foundation Design of Buildings and Other Structures (Except Hydraulic Structures*, Technical Report No. 3-526, Department of the Army, Corps of Engineers.
Figures 7-1 7-2 7-4	Janbu, N., *Stability Analysis of Slopes with Dimensionless Parameters*, Harvard Soil Mechanics Series No. 46, Harvard University, Cambridge, Mass.
Figure 7-7	Jakobsen, R. E., *The Design of Embankments on Soft Clays*, Geotechnique, 1948, Institution of Civil Engineers, London.

Figure or Table	*Acknowledgment*
Figures 8-2 8-3	Marsland, A., *Model Experiments To Study the Influence of Seepage on the Stability of a Sheeted Excavation in Sand,* Geotechnique, 1952-1953, Institution of Civil Engineers, London.
Figures 8-5 8-6 (Top Panel)	Barber, E. W., *Subsurface Drainage of Highways*, Highway Research Board Bulletin 209, Highway Research Board, Washington, D. C.
Figure 8-6 (Bottom Panel)	Cedergren, H. R., *Seepage Requirements of Filters and Pervious Bases*, Journal of the Soil Mechanics and Foundations Division, October 1960, American Society of Civil Engineers, New York, N.Y.
Figure 8-7	Kirkham, D., *Seepage into Ditches from a Plane Water Table Overlying a Gravel Substratum*, Journal of Geophysical Research, April 1960, American Geophysical Union, Washington, D. C.; Kirkham, D., *Seepage into Ditches in the Case of a Plane Water Table and an Impervious Substratum*, Transactions, American Geophysical Union, June 1950, Washington, D. C.
Figure 8-9 (Bottom Panel)	Avery, S. B., *Analysis of Groundwater Lowering Adjacent to Open Water*, Vol. 77, Proceedings, American Society of Civil Engineers, New York.
Figure 8-10	*Soil Mechanics Design, Seepage Control*, Chapter I, Part CXIX, Engineering Manual, Civil Works Construction, Department of the Army, Corps of Engineers.
Figure 9-1	*Design of Small Dams*, U. S. Department of the Interior, Bureau of Reclamation.
Table 9-2	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 10-3 10-4	Caquot, A., and Kerisel, J., *Tables for the Calculation of Passive Pressure, Active Pressure and Bearing Capacity of Foundations*, Gauthier-Villars, Paris.
Figure 10-5 (Bottom Panel)	Richart, F. E., and Schmertmann, J., *The Effect of Seepage on the Stability of Sea Walls,* Proceedings, First Conference on Coastal Engineering, University of Florida, Gainesville, Fla.
Figure 10-5 (Middle Panel)	Gray, H., *Contribution to the Analysis of Seepage Effects in Backfills*, Geotechnique, 1958, Institution of Civil Engineers, London.
Figure 10-6	Based on work by Terzaghi, K., *Anchored Bulkheads,* Transactions, American Society of Civil Engineers, Paper No. 2720, Vol. 119, 1954.
Figures 10-9 10-10	Terzaghi, K., and Peck, R. B., *Soil Mechanics in Engineering Practice*, John Wiley & Sons, Inc., New York, N.Y.
Figure 10-11	*Concrete Information No St 46,* Concrete Crib Retaining Walls, Portland Cement Association, May 1952, Chicago, Ill.
Figure 10-13 (Upper Panel)	Rowe, P. W., *Anchored Sheet Pile Walls*, Proceedings, Institution of Civil Engineers, January 1952, London.
Figures 11-1 11-2	Meyerhof, G. G., *Influence of Roughness of Base and Ground Water Condition on the Ultimate Bearing Capacity of Foundations,* Geotechnique, 1955, Institution of Civil Engineers, London.
Figure 11-3	Meyerhof, G. G., *The Bearing Capacity of Foundations Under Eccentric and Inclined Loads,* Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Berichthaus, Zurich.
Figure 11-4	Meyerhof, G. G., *The Ultimate Bearing Capacity of Foundations on Slopes*, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, Butterworths Scientific Publications, London.
Figure 11-5	Button, S. J., *The Bearing Capacity of Footings on a Two-layer Cohesive Subsoil*, Proceedings, Third International Conference on Soil Mechanics and Foundation Engineering, Berichthaus, Zurich.

Figure or Table	*Acknowledgment*
Figure 11-9 (Upper Panel)	Fox, E. N., *The Mean Elastic Settlement of a Uniformly Loaded Area at a Depth Below the Ground Surface*, Proceedings, Second International Conference on Soil Mechanics and Foundation Engineering.
Figures 11-13, 11-14	Hetenyi, M., *Beams on Elastic Foundation,* The University of Michigan Press, Ann Arbor, Michigan.
Figure 12-1 (Upper Panel)	Skempton, A. W., *The Bearing Capacity of Clays,* Proceedings, Building Research Congress, 1951, London.
Figure 12-2	Meyerhof, G. G., *The Ultimate Bearing Capacity of Foundations,* Geotechnique, 1950, Institution of Civil Engineers, London.
Figure 13-1 (Upper Panel)	Tomlinson, M. J., *The Adhesion of Piles Driven in Clay Soils*, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, Butterworths Scientific Publications, London.
Figure 13-2	Meyerhof, G. G., *Compaction of Sands and Bearing Capacity of Piles*, Journal of the Soil Mechanics and Foundations Division, December 1959, American Society of Civil Engineers, New York, N. Y.
Figures 13-4, 13-5, 13-6	Reese, L. C., and Matlock H., *Nondimensional Solutions for Laterally Loaded Piles With Soil Modulus Assumed Proportional to Depth*, Proceedings, Eighth Texas Conference on Soil Mechanics and Foundation Engineering, University of Texas, Austin, Texas.
Figure 13-7 (Upper Panel)	Whitaker, T., *Experiments with Model Piles in Groups*, Geotechnique, 1957, Institution of Civil Engineers, London.
Figures 16-1, 16-4, 16-5	Richart, F. E., Jr., *Foundation Vibrations*, Journal of the Soil Mechanics and Foundations Division, August 1960, American Society of Civil Engineers, New York, N. Y.

CHAPTER 1. SOIL CLASSIFICATION

Section 1. INTRODUCTION

1. **SCOPE.** This chapter presents criteria for soil identification and classification plus information on soil deposits and their geographical distribution in the United States. These subjects are not concerned directly with design criteria, but form the basis for application of soil mechanics in foundation design.

2. **CANCELLATION.** This revised edition of *Soil Mechanics, Foundations, and Earth Structures*, NAVFAC DM-7, cancels and supersedes *Soil Mechanics, Foundations, and Earth Structures*, NAVDOCKS DM-7, in its entirety, and all subsequent changes.

Section 2. PRINCIPAL SOIL DEPOSITS

1. **GEOLOGICAL ORIGINS.** For civil engineering purposes, soil is defined as the residual or transported product of rock decay. Soil may be excavated without blasting and is penetrated in borings by ordinary soil sampling equipment.

a. Geological Classification. Soil deposits are usually described in general terms according to the geological process involved in their formation. On this basis there are four principal categories plus secondary types whose characteristics are summarized in Table 1-1.

b. Importance. For quantitative foundation analysis, a geological description is inadequate and a more specific classification is required. However, a geological description assists in correlating experiences between several sites and, in a general sense, indicates the pattern of strata to be expected. 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.

2. **REGIONAL DISTRIBUTION IN THE UNITED STATES.** See Figure 1-1 (Soil Map) for distribution.

a. Major Soil Division. Figure 1-1 indicates the four principal soil types in the United States. Locations containing insignificant soil cover are designated as "Nonsoil areas." The map boundaries necessarily omit many detailed exceptions to the overall pattern. Dominant land form in any area is related to the principal soil deposit and geological processes that formed the deposit. See Table 1-2 for general subsurface and physiographic features of areas numbered on Figure 1-1.

b. Special Soil Divisions.

(1) Soil Types Available. Practically all states contain soils representative of more than one major geological origin. The Southern Cascade Mountains in Oregon and the Hawaiian Island Group contain the principal concentrations of volcanic deposits. Almost all soil types of engineering significance are present within the United States.

(2) Soil Types Not Available. Two materials of great importance in other countries are infrequently encountered in the United States. One is the laterite group, residual material rich in alumina and

TABLE 1-1
Classification of Principal Soil Deposits According to Origin

Geological origin	Process involved in formation	Nature of deposits	Typical gradation
Residual.	Soil weathered in place from parent rocks with little or no alteration by transport.	Almost invariably becomes more compact, rockier, and less weathered with increasing depth. May reflect alternation of hard and soft layers or stratification of parent rock if weathering is incomplete.	Product of complete weathering is clay of a type depending on the weathering process and parent rock, plus varying amounts of resistant silica particles. Soil at intermediate stage reflects composition of parent rock.
Alluvial .	Materials transported and redeposited by action of water.	Usually with pronounced stratification. Typical river deposit consists of fine grained material of recent origin overlying coarser strata dating from earlier stage of river development.	Ranges from finest grained lacustrine or marine clays to very coarse gravel, cobble or boulders in alluvial fan or stream terrace deposits.
Glacial . .	Materials transported and redeposited by glacial ice or by melt waters flowing from glaciers.	Stratification varies greatly according to deposit, from heterogeneous moraines and till to finely stratified (varved) silt and clay in glacial lakes, or irregularly layered ice contact deposits and outwash plains.	Till and moraines are typically of broad gradation ranging from clay to boulders. Grain size in outwash generally decreases with distance from source of melt water.
Loessial.	Soil transported by wind without subsequent redeposition.	In loess, horizontal stratification is indistinct or nonexistent except for weathered horizons. Frequently has secondary structure of vertical cracks, joints, and root holes.	Most uniform in gradation of all principal soil types. Loess range from clayey silt to silty fine sand. Dune sands generally fine to medium size lacking silt or clay.
Secondary geological origins.	Organic soils formed in place by growth and decay of plants.	Most U.S. peats formed as filled basin deposits in irregular glacial topography or in areas of subsidence on southern and eastern coasts.	Dark colored, finely divided peats are product of advanced decomposition in the presence of air. Fibrous peat has been continuously submerged.
Do	Ash and pumice deposited by volcanic action.	Frequently associated with lava flows and mud flows, or may be mixed with nonvolcanic sediments.	Typically shardlike particles of silt size with larger volcanic debris. Weathering and redeposition produces highly plastic clay.
Do	Materials precipitated or evaporated from solutions of high salt content.	Includes such varieties as oolites precipitated from calcium in sea water or evaporites formed in playa lakes under arid conditions.	May form cemented soils or soft sedimentary rocks including gypsum or anhydrite.

iron oxide formed in tropical regions. The other is the uplifted, leached, and highly sensitive marine clay that is limited in the United States to the St. Lawrence River Valley.

3. **LOCAL SOIL AREAS.** Foundations for large or deep structures usually reach soils of the principal geological type within an area, but roads, airfields, or foundations of small or shallow structures may involve materials differing from the main soil type.

a. **Alluvial Deposits.** In any area, surface materials near drainage channels frequently are of alluvial origin. Waterfront locations, in particular, may contain recent shallow sediments and artificial fills that differ from the principal geological type of the area.

b. **Organic Deposits.** Organic deposits are common in low-lying or waterlogged terrain. Irregular topography resulting from glaciation, and subsidence of the southern and eastern coastal plane provided numerous sites for formation of peat. These deposits are not distinguished separately in Figure 1-1, but are of major importance in foundation engineering in many locations.

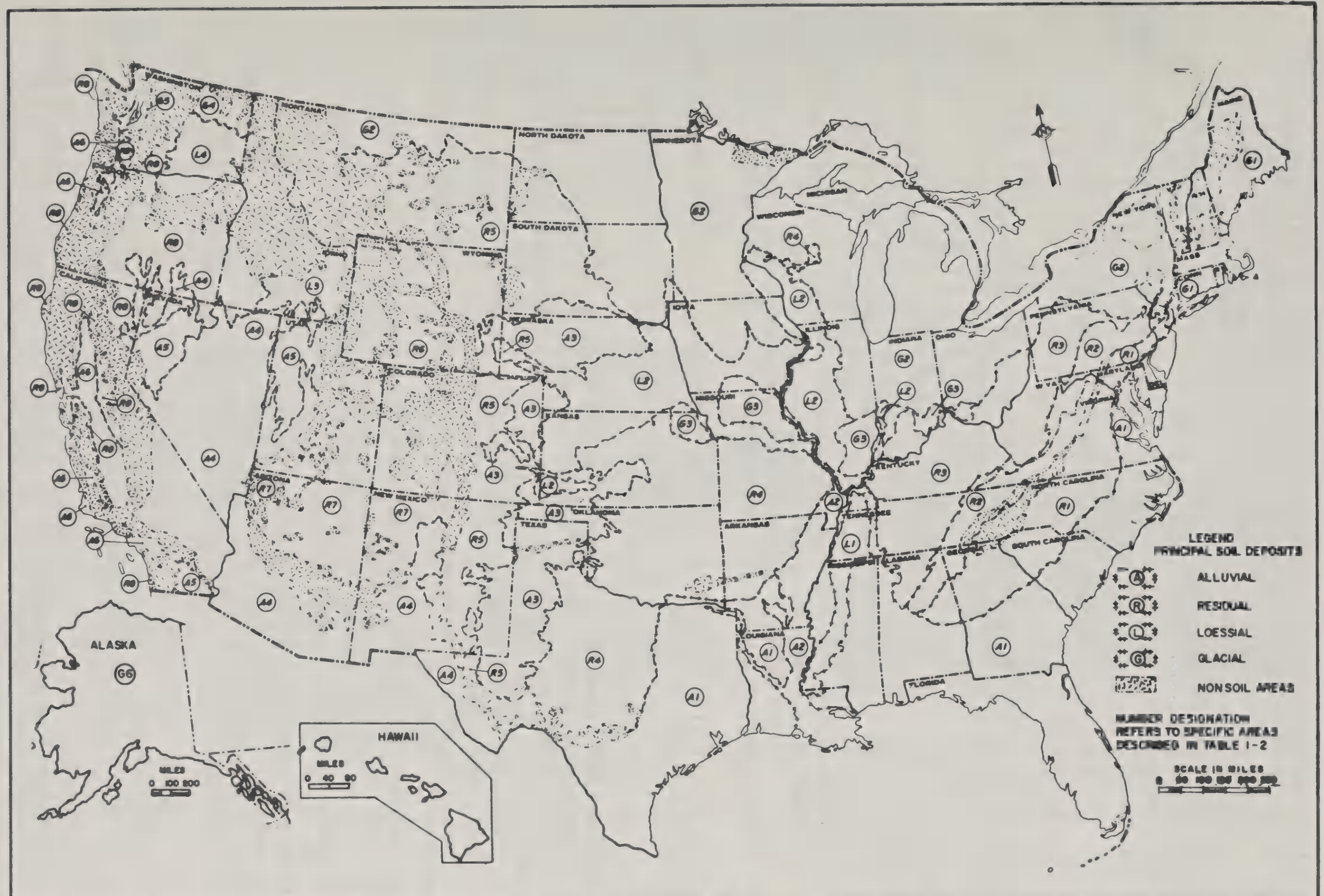

FIGURE 1-1
Distribution of Principal Soil Deposits in the United States

c. Surface Alteration. Investigations for highway and airfield pavements may be concerned with surficial "A" and "B" horizons that have been altered from the original soils.

(1) "A" Horizon. The surface "A" horizon is rich in humus and has undergone leaching, oxidation, and chemical weathering plus removal of clay particles by mechanical migration.

(2) "B" Horizon. The underlying "B" horizon has received products of weathering and leaching from above and is enriched in clay from the surface layer.

(3) "C" Horizon. Beneath the "B" horizon is the unaltered parent soil, which is designated as the "C" horizon.

Section 3. SOIL IDENTIFICATION

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

a. Field Classification. Identify constituent materials visually by their gradation or plasticity characteristics.

(1) Coarse Grained Soils. Coarse grained soils, those with sand, gravel, or cobbles predominating, are classified according to grain size.

(2) Fine Grained Soils. Fine grained soils cannot be divided between silt and clay by visual identification of grain size, but are distinguishable by plasticity characteristics.

(a) 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.

(b) 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.

b. 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.

(4) Condition of individual grains in coarse grained soils, such as their angularity, cementation, surface coating, and hardness of particles.

c. Compactness or Hardness. Estimate consistency in situ by measuring resistance to penetration of a selected penetrometer or sampling device. Ordinarily, 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 spoon sample 1 foot into undisturbed soil in a boring. The number of blows per foot thus obtained is known as the standard penetration resistance.

(1) Descriptive Terms. See Table 1-3 for descriptive terms of compactness or hardness based on standard penetration resistance. Penetration resistance of coarse grained soils depends on both density and overburden pressure acting at depth sampled so that terms of compactness rather than density are appropriate.

(2) Fine Grained Soils. Use of pocket penetrometer calibrated to unconfined compressive strength to check the hardness of fine grained soils.

(3) Routine Methods. These methods do not provide precise values of soil consistency in situ, but they should be made routine in exploration work.

TABLE 1-2
Distribution of Principal Soil Deposits in the United States

Origin of principal soil deposits	Symbol for area, Fig. 1-1	Physiographic province	Physiographic features	Characteristic soil deposits
Alluvial	A1	Coastal Plain	Terraced or belted coastal plain with submerged border on Atlantic. Marine plain with sinks, swamps, and sand hills in Florida.	Marine and continental alluvium thickening seaward. Organic soils on coast. Broad clay belts west of Mississippi. Calcareous sediments on soft and cavitated limestone in Florida.
Do	A2	Mississippi Alluvial Plain	River flood plain and delta	Recent alluvium, fine grained and organic in low areas, overlying clays of Coast Plain.
Do	A3	High Plains Section of Great Plains Province.	Broad intervalley remnants of smooth fluvial plains.	Outwash mantle of silt, sand, silty clay, lesser gravels, underlain by soft shale, sandstone, and marls.
Do	A4	Basin and Range Province	Isolated ranges of dissected block mountains separated by desert plains.	Desert plains formed principally of alluvial fans of coarse grained soils merging to playa lake deposits. Numerous nonsoil areas.
Do	A5	Major Lakes of Basin and Range Province.	Intermontane Pleistocene lakes in Utah and Nevada, Salton Basin in California.	Lacustrine silts and clays with beach sands on periphery. Widespread sand areas in Salton Basin.
Do	A6	Valleys and Basins of Pacific Border Province.	Intermontane lowlands, Central Valley, Los Angeles Basin, Willamette Valley.	Valley fills of various gradations, fine grained and sometimes organic in lowest areas near drainage system.
Residual	R1	Piedmont Province	Dissected peneplain with moderate relief. Ridges on stronger rocks.	Soils weathered in place from metamorphic and intrusive rocks (except red shale and sandstone in New Jersey). Generally more clayey at surface.
Do	R2	Valley and Ridge Province	Folded strong and weak strata forming successive ridges and valleys.	Soils in valleys weathered from shale, sandstone, and limestone. Soil thin or absent on ridges.
Do	R3	Interior Low Plateaus and Appalachian Plateaus.	Mature, dessicated plateaus of moderate relief.	Soils weathered in place from shale, sandstone, and limestone.
Do	R4	Ozark Plateau, Ouachita Province, Portions of Great Plains and Central Lowland, Wisconsin Driftless Section.	Plateaus and plains of moderate relief, folded strong and weak strata in Arkansas.	Soils weathered in place from sandstone and limestone predominantly, and shales secondarily. Numerous nonsoil areas in Arkansas.
Do	R5	Northern and Western Sections of Great Plains Province.	Old plateau, terrace lands, and Rocky Mountain Piedmont.	Soils weathered in place from shale, sandstone, and limestone including areas of clay-shales in Montana, South Dakota, Colorado.
Do	R6	Wyoming Basin	Elevated plains	Soils weathered in place from shale, sandstone, and limestone.
Do	R7	Colorado Plateaus	Dissected plateau of strong relief	Soils weathered in place from sandstone primarily, shale and limestone secondarily.
Do	R8	Columbia Plateaus and Pacific Border Province.	High plateaus and piedmont	Soils weathered from extrusive rocks in Columbia Plateaus and from shale and sandstone on Pacific Border. Includes area of volcanic ash and pumice in Central Oregon.
Loessial	L1	Portion of Coastal Plain	Steep bluffs on west limit with incised drainage.	30 to 100 ft of loessial silt and sand overlying Coastal Plain alluvium. Loess cover thins eastward.

TABLE 1-2 (Continued)

Distribution of Principal Soil Deposits in the United States

Origin of principal soil deposits	Symbol for area,[a] Fig. 1-1	Physiographic province	Physiographic features	Characteristic soil deposits
Do	L2	Southwest Section of Central Lowland. Portions of Great Plains.	Broad intervalley remnants of smooth plains.	Loessial silty clay, silt, silty fine sand with clayey binder in western areas, calcareous binder in eastern areas.
Do	L3[f]	Snake River Plain of Columbia Plateaus.	Young lava plateau	Relatively thin cover of loessial silty fine sand overlying fresh lava flows.
Do	L4	Walla Walla Plateau of Columbia Plateaus.	Rolling plateau with young incised valleys.	Loessial silt as thick as 75 ft overlying basalt. Incised valleys floored with coarse grained alluvium.
Glacial ...	G1	New England Province.....	Low peneplain maturely eroded and glaciated.	Generally glacial till overlying metamorphic and intrusive rocks, frequent and irregular outcrops. Coarse, stratified drift in upper drainage systems. Varved silt and clay deposits at Portland, Boston, New York, Connecticut River Valley, Hackensack area.
Do	G2	Northern Section of Appalachian Plateau, Northern Section of Central Lowland.	Mature glaciated plateau in northeast, young till plains in western areas.	Generally glacial till overlying sedimentary rocks. Coarse stratified drift in drainage system. Numerous swamps and marshes in north central section. Varved silt and clay deposits at Cleveland, Toledo, Detroit, Chicago, northwestern Minnesota.
Do	G3	Areas in Southern Central Lowland.	Dissected old till plains	Old glacial drift, sorted and unsorted, deeply weathered, overlying sedimentary rocks.
Do	G4	Western area of Northern Rocky Mountains.	Deeply dissected mountain uplands with intermontane basins extensively glaciated.	Varved clay, silt, and sand in intermontane basins, overlain in part by coarse grained glacial outwash.
Do	G5	Puget Trough of Pacific Border Province.	River valley system, drowned and glaciated.	Variety of glacial deposits, generally stratified, ranging from clayey silt to very coarse outwash.
Do	G6	Alaska Peninsula	Folded mountain chains of great relief with intermontane basins extensively glaciated.	In valleys and coastal areas widespread deposits of stratified outwash, moraines and till. Numerous nonsoil areas.
		Hawaiian Island Group.....	Coral islands on the west, volcanic islands on the east.	Coral islands generally have sand cover. Volcanic ash, pumice, and tuff overlie lava flows and cones on volcanic islands. In some areas volcanic deposits are deeply weathered.
Nonsoil areas		Principal mountain masses .	Mountains, canyons, scablands, badlands.	Locations in which soil cover is very thin or has little engineering significance because of rough topography or exposed rock.

Section 4. UNIFIED SOIL CLASSIFICATION SYSTEM

1. **REFERENCE.** Soil designations in this manual conform to the Unified Soil Classification MIL-STD-619B (ME), 12 June 1968, which was modified from the former Airfield Classification in 1952 for adoption by the U.S. Corps of Engineers (USCE) and the U.S. Bureau of Reclamation (USBR).

2. **UTILIZATION.** Classify soils in accordance with the Unified System and include the appropriate group symbol in soil descriptions. See Table 1-4 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.

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

b. **Silts and Clays.** Fine grained soils are identified according to plasticity characteristics determined in Atterberg limit tests. Categories are illustrated on the plasticity chart in Figure 3-1.

c. **Organic Soils.** Materials containing vegetal 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.

Section 5. ROCK CLASSIFICATION AND DESCRIPTION

1. **REFERENCE.** Rock classification includes (a) classification of constituents, (b) description of appearance, and (c) indication of hardness and degree of cementation. An accurate description of the rock encountered in subsurface exploration work will provide the designer with the information necessary to design the substructure, and at the same time, will tend to reduce contract costs by providing contractors with reliable subsurface information on which they can estimate excavation work. Rock designations in this Manual are given in Table 1-5 and are intended as a guide for the classification and description of rock encountered in subsurface exploration.

TABLE 1-3
Description of Soil Compactness or Consistency

Primary soil type	Compactness or consistency	Range of standard penetration resistance[1]	Range of unconfined compressive strength
Coarse grained soils. (More than half of material is larger than No. 200 sieve size.)	Very loose	Less than 4 blows per foot.	Not applicable.
	Loose	4 to 10	Do.
	Medium compact	10 to 30	Do.
	Compact	30 to 50	Do.
	Very compact..	Greater than 50	Do.
Fine grained soils. (More than half of material is smaller than No. 200 sieve size.)	Very soft.....	Less than 2 blows per foot.	Less than 0.25 tsf
	Soft	2 to 4	0.25 to 0.5.
	Medium stiff...	4 to 8	0.5 to 1.0.
	Stiff..........	8 to 15	1.0 to 2.0.
	Very stiff	15 to 30	2.0 to 4.0.
	Hard	Greater than 30	Greater than 4.0.

[1]Number of blows of 140 lb weight falling 30 in. to drive 2-in.-OD, 1-3/8-in.-ID, sampler 1 ft.

TABLE 1-4
Unified Soil Classification System

Primary divisions			Group symbol	Secondary divisions	Laboratory classification criteria		Supplementary criteria for visual identification
Coarse grained soils. (More than half of material is larger than No. 200 sieve size.)	Gravels. (More than half of the coarse fraction is larger than No. 4 sieve size.)	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 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3		Wide range in grain size and substantial amounts of all intermediate particle sizes.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.	Not meeting all gradation requirements for GW.		Predominantly one size or a range of sizes with some intermediate sizes missing.
	. . . do	Gravels with fines. (More than 12% of material smaller than No. 200 sieve size.)[1]	GM	Silty gravels, and gravel-sand-silt mixtures, which may be poorly graded.	Atterberg limits below "A" line, or PI less than 4[2]	Atterberg limits above "A" line with PI between 4 and 7 is borderline case GM-GC	Nonplastic fines or fines of low plasticity.
			GC	Clayey gravels, and gravel-sand-clay mixtures, which may be poorly graded.	Atterberg limits above "A" line, with 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 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3		Wide range in grain sizes and substantial amounts of all intermediate particle sizes.
			SP	Poorly graded sands, gravelly sands, little or no fines.	Not meeting all gradation requirements for SW		Predominately one size or a range of sizes with some intermediate sizes missing.
	. . . do	Sands with fines. (More than 12% of material smaller than No. 200 sieve size.)[1]	SM	Silty sands, and sand-silt mixtures, which may be poorly graded.	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, and sand-clay mixtures, which may be poorly graded.	Atterberg limits above "A" line, with PI greater than 7		Plastic fines.

TABLE 1-4 (Continued)
Unified Soil Classification System

Primary divisions		Group symbol	Secondary divisions	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.)	Silts and clays. (Liquid limit less than 50.)	ML	Inorganic silts, clayey silts, rock flour, silty very 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; silty, sandy or gravelly 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
Do ...	Silts and clays. (Liquid limit greater than 50.)	MH	Inorganic silts, clayey silts, elastic silts, micaceous or diatomaceous silty or fine sandy soils.	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 and silty clays of medium to high plasticity.	Atterberg limits below "A" line		Medium to high	None to very slow	Slight to medium
	Highly organic soils ...	Pt	Peat, meadow mat, highly organic soils.	High ignition loss, LL and PI decrease after drying		Organic color and odor, spongy feel, frequently fibrous texture.		

[1]Materials with 5 to 12 percent smaller than No. 200 sieve are borderline cases, designated: GW-GM, SW-SM.
[2]See Ch. 3, Figure 3-1, for position on plasticity chart.

2. UTILIZATION. Records of rock explorations shall be shown by symbol and by descriptive information according to Table 1-5. The descriptive information may be presented either by word description or by numerals, with a legend explaining the numerals, the former being preferable. Particular attention should be given to a clear presentation of those characteristics of rocks that will convey to the contractor accurate information on the excavation problems so he can determine what excavation methods and procedures will be involved. Complicated descriptions of scientific interest not pertinent to the excavation problem shall not be used.

Symbol	Rock classifica
	SANDSTONE
	CONGLOMER
	COAL
	COMPACTIO SHALE
	CEMENTED SHALE
	INDURATED CLAY
	LIMESTONE
	CHALK (OR MARL)
	CORAL
	GNEISS
	SCHIST
	GRAYWACKE
	QUARTZITE
	DOLOMITE
	MARBLE
	SOAPSTONE & SERPENTI
	SLATE
	GRANITE
	DIORITE
	GABBRO
	RHYOLITE
	ANDESITE
	BASALT (TRAP)
	TUFF OR TUFF BRECC
	AGGLOMERAT OR FLOW BRE

fingernail
atched easily
hed with

ith knife
hed with knife

a. Open

nted) b. Cemented or tight

Example

* The physical properties of the rock may be designated on the boring logs either by word descriptions or by using the appropriate number or numbers shown here for the applicable properties. Thus, the log for a core hole may be indicated on engineering drawings in the following manner:

LOG EXAMPLE

	COMPACTION SHALE	12, 16, 28a, 34, 43, 46
	LIMESTONE	2, 9, 17, 22, 27a, 34, 36
	CEMENTED SHALE	13, 17, 27b, 33, 42, 45
	COAL	17, 33, 45
	CLAY	1, 15, 33
	SANDSTONE	4, 9, 17, 21, 23, 27b, 33, 36

Where numbers are used on the logs to indicate the physical properties, a key to the numbers must be included on each sheet of log drawings.

Such data as core losses or percentages of core recovery, depths of drill-water loss, pressure-test results, and water-level measurements obtained in connection with drilling operations also should be included on the logs as plotted on engineering drawings. Water-level measurements should always be accompanied by the dates on which the measurements were made. The logs also should show the dates on which the holes were drilled.

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TABLE 1-5
Guide for Classification and Description of Rocks With Emphasis on Their Engineering Properties

Symbol	Rock classification	Applicable descriptive property numbers*
	SANDSTONE	
	CONGLOMERATE	
	COAL	
	COMPACTION SHALE	
	CEMENTED SHALE	
	INDURATED CLAY	
	LIMESTONE	
	CHALK (OR MARL)	
	CORAL	
	GNEISS	
	SCHIST	
	GRAYWACKE	
	QUARTZITE	
	DOLOMITE	
	MARBLE	
	SOAPSTONE & SERPENTINE	
	SLATE	
	GRANITE	
	DIORITE	
	GABBRO	
	RHYOLITE	
	ANDESITE	
	BASALT (TRAP)	
	TUFF OR TUFF BRECCIA	
	AGGLOMERATE OR FLOW BRECCIA	

FOR USE OF THIS COLUMN SEE EXAMPLE TO RIGHT

Word descriptions generally are preferable to use of property numbers

General comments

The 25 rocks shown on this plate represent the principal rock types encountered. Petrologic variations from these common rock forms that have no engineering significance have been omitted intentionally. Thus, while syenite or pegmatite are recognized by geologists as differing in certain lithologic respects from granite, to the engineer these differences are not of sufficient practical significance to warrant use of any term but "granite" in identifying these rocks on engineering drawings. On the other hand, diorite and gabbro are included because, in certain sections of the country, these are recognized as individual rock types, particularly in dimension stone and aggregate quarries. The rock symbols shown on the chart have been standardized by the U.S. Geological Survey and have been used widely by other United States and foreign agencies. Consequently, when symbols are used to differentiate rocks on boring logs or geological cross sections, only the symbols appearing on this chart shall be used. If a rock such as trachyte is identified on boring logs, the symbol representing the most closely related rock type on the chart, rhyolite in this case, shall be used.

Instructions governing use of descriptive properties

This chart is issued as a standard guide for describing rocks excavated for foundations or cut slopes. It is not to be considered as amending petrographic classification requirements for concrete aggregate investigations or riprap. This portion of the Manual is issued in recognition of the need for uniform and more truly representative descriptions of bedrock conditions disclosed by drilling operations. For logs of rock borings to present information that will simultaneously guide the structural design engineer and assist prospective contractors in appraising the excavation properties of rocks, the lithologic and physical properties encountered by drilling or other explorations will be enumerated in the column to the right of the rock symbol, either by word description or by the applicable property number shown on this chart. Generally speaking, the use of word descriptions is preferable to the use of property numbers.

Key to physical properties of rocks

Bedding Characteristics –
1. Massive
2. Thin to med. bedded
3. Fissile
4. Crossbedded
5. Foliated
6. Platy
7. Fragmental

Lithologic Characteristics –
8. Clayey
9. Shaly
10. Calcareous (limy)
11. Siliceous
12. Sandy
13. Silty
14. Plastic seams

Hardness and Degree of Cementation –
15. Very soft or plastic
16. Soft – Can be scratched with fingernail
17. Moderately hard – Can be scratched easily with knife; cannot be scratched with fingernail
18. Hard – Difficult to scratch with knife
19. Very hard – Cannot be scratched with knife
20. Poorly cemented
21. Cemented

Texture –
22. Dense
23. Fine
24. Medium
25. Coarse

Structure –
26. Bedding a. Flat; b. Gently dipping; c. Steeply dipping
27. Fractures, scattered
28. Fractures, closely spaced
29. Brecciated (sheared & fragmented)

(27–29: a. Open; b. Cemented or tight)

30. Joints
31. Faulted
32. Slickensides

Degree of Weathering –
33. Unweathered
34. Slightly weathered
35. Badly weathered

Solution and Void Conditions –
36. Solid, contains no voids
37. Vuggy (pitted)
38. Vesicular
39. Porous
40. Cavities
41. Cavernous

Swelling Properties –
42. Nonswelling
43. Swelling

Slaking Properties –
44. Nonslaking
45. Slakes slowly on exposure
46. Slakes readily on exposure

Example

* The physical properties of the rock may be designated on the boring logs either by word descriptions or by using the appropriate number or numbers shown here for the applicable properties. Thus, the log for a core hole may be indicated on engineering drawings in the following manner:

LOG EXAMPLE

	COMPACTION SHALE	12, 16, 28a, 34, 43, 46
	LIMESTONE	2, 9, 17, 22, 27a, 34, 36
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	COAL	17, 33, 45
	CLAY	1, 15, 33
	SANDSTONE	4, 9, 17, 21, 23, 27b, 33, 36

Where numbers are used on the logs to indicate the physical properties, a key to the numbers must be included on each sheet of log drawings.

Such data as core losses or percentages of core recovery, depths of drill-water loss, pressure-test results, and water-level measurements obtained in connection with drilling operations also should be included on the logs as plotted on engineering drawings. Water-level measurements should always be accompanied by the dates on which the measurements were made. The logs also should show the dates on which the holes were drilled.

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CHAPTER 2. EXPLORATION AND SAMPLING

Section 1. INTRODUCTION

1. **SCOPE.** This chapter contains information on exploration methods and criteria for test boring and sampling.

2. **RELATED CRITERIA.** Other criteria related to exploration and sampling appear elsewhere in this DM series. See the following sources:

Subject	*Sources*
Exploration for airfields.	NAVFAC DM-21
Exploration for highways	NAVFAC DM-5

3. **EXPLORATION PROGRAMS.** Subsurface investigation has three phases; reconnaissance, preliminary exploration, and detailed exploration.

a. **Reconnaissance.** Reconnaissance includes a review of available topographic and geological information, aerial photographs, and data from previous investigations, and site examination. Geophysical methods are applicable in special cases. Reconnaissance establishes the number and locations of preliminary borings.

b. **Preliminary Exploration.** This includes borings to recover samples suitable for identification tests only. Even if a more detailed program will follow, obtain samples and boring logs of such quality that results may be incorporated with the data from the final investigation.

c. **Detailed Exploration.** This is based on the results of previous phases and generally includes recovery of undisturbed samples for structural properties tests and, in some cases, field tests and observations.

Section 2. EXISTING SOIL AND GEOLOGICAL MAPS

1. **SOURCES.** Data on the physical geology of the United States are available in maps and reports by government agencies and professional societies. (See Table 2-1.) Assemble pertinent data from these publications for use in site reconnaissance before undertaking any boring program.

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, ground water 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.

TABLE 2-1
Sources of Geological Information

Series	Description of material
U.S. Geological Survey (USGS) ..	Consult *USGS Index to Publications* from Superintendent of Documents, Washington, D. C. Order publications from Superintendent of Documents. Order maps from USGS, Washington, D. C.
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 and includes 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 mineral and petroleum resources. Areal and bedrock geology maps for specific locations included in many publications.
Water supply papers	Series includes papers on ground water resources in specific localities and are generally accompanied by description of subsurface conditions affecting ground water plus observations of ground water levels.
Topographic maps.............	Topographic contour maps in all states, widespread coverage being continually expanded.
U.S. Coast and Geodetic Survey (USC&GS).	Consult Index from Director, U.S. Coast and Geodetic Survey, Washington, D. C.
Nautical Charts...............	Charts of coastal areas showing available soundings of sea bottom plus topographic and cultural features adjacent to the coast or waterways.
U.S. Department of Agriculture (USDA), Soil Conservation Service.	Consult Highway Research Board (HRB), Bull. No. 22-R, *Agricultural Soil Maps, Status July 1957*, for coverage by counties of USDA Soil Maps and Reports.
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 geologists bulletins, reports, and maps.	Consult HRB Bull. No. 180, *Geologic Survey Mapping in the United States*, for addresses of all state geological organizations. Most states provide excellent detailed local geological maps and reports covering specific areas or features in the publications of the state geologists.
Geological Society of America (GSA).	Write for index to GSA, 419 West 117 Street, New York, N. Y.
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.

Section 3. AIR PHOTO INTERPRETATION

1. **SOURCES.** Aerial photographs at scales of 1:20,000 and 1:40,000 are complete for almost the entire United States. For areas of special interest, mosaics have been assembled from individual pictures.

a. **Sponsoring Agencies.** These are primarily organizations of the Federal Government, including U.S. Department of Agriculture (USDA), U.S. Geological Survey (USGS), U.S. Bureau of Reclamation (USBR), U.S. Coast and Geodetic Survey (USC&GS), Department of Defense (DOD), and Tennessee Valley Authority (TVA).

b. **Coverage of Aerial Photographs.** Consult maps, "Status of Aerial Mosaics" and "Status of Aerial Photographs," (latest revisions) available from Chief of Distribution, USGS. When ordering pictures from source agencies, specify whether stereoscopic sets or mosaics are required.

c. Photo Interpretation Services. These services are available at Photographic Interpretation Center, Anacostia Naval Air Station, District of Columbia and Air Navigations Offices.

2. UTILIZATION. Use of photographs and mosaics is routine in highway and airfield work. For unfamiliar sites they aid in planning and layout of an appropriate boring program.

a. Flight Strips. Most aerial photographs are taken as flight strips with 60 percent or more overlap between pictures.

b. Interpretation. When overlapping pictures are viewed stereoscopically, an exaggerated ground relief appears. From the appearance of land forms or erosional or depositional features, the character of soil or rock may be interpreted.

3. LIMITATIONS. Interpretation of aerial photographs requires considerable experience, and results obtained depend on the interpreter's proficiency.

a. Accuracy. Aerial photographs necessarily deal with surface and near-surface conditions and accuracy is limited where dense vegetation obscures ground features.

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 foundation analysis.

Section 4. GEOPHYSICAL METHODS

1. UTILIZATION. See Table 2-2 for geophysical methods and applications.

a. Advantages. In contrast to borings, geophysical surveys explore large areas or projects of great linear extent rapidly and economically. They indicate average conditions in the proximity of a test setup, rather than along the restricted vertical line of a boring. This helps detect irregularities of bedrock surface or interface between strata.

TABLE 2-2
Geophysical Methods of Exploration

Name of method	Procedure or principle utilized	Applicability
Seismic methods: . . .	Based on time required for seismic waves to travel from source of blast to points on ground surface, as measured by geophones spaced at intervals on a line at the surface.	
Refraction	Refraction of seismic waves at the interface between different strata gives a pattern of arrival times vs distance at a line of geophones.	Utilized to determine depth to rock or other lower stratum substantially different in wave velocity than the overlying material. Generally limited to depths up to 100 ft of a single stratum. Used only where wave velocity in successive layers becomes greater with depth.
Reflection	Geophones record travel time for the arrival of seismic waves reflected from the interface of adjoining strata.	Suitable for determining depths to deep rock strata. Generally applied only for depths exceeding 2,000 ft. Reflected impulses are weak and easily obscured by the direct surface and shallow refraction impulses.

TABLE 2-2 (Continued)
Geophysical Methods of Exploration

Name of method	Procedure or principle utilized	Applicability
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 ft of subsurface strata. Principal applications for investigating foundations of dams and other large structures, particularly in exploring granular river channel deposits or bedrock surfaces.
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.
Continuous vibration method.	The travel time of transverse or shear waves generated by a mechanical vibrator consisting of a pair of eccentrically weighted disks is recorded by seismic detectors placed at specific distances from the vibrator.	Velocity of wave travel and natural period of vibration gives some indication of soil type. Travel time plotted as a function of distance indicates depths or thicknesses of surface strata. Useful in determining dynamic modulus of subgrade reaction and obtaining information on the natural period of vibration for design of foundations of vibrating structures.
Magnetic measurements.	Magnetometer is used to measure the vertical component of the earth's magnetic field at closely spaced stations in an area.	Difficult to interpret in quantitative terms but indicates the outline of intrusive igneous dikes or other large igneous masses. Infrequently used in civil engineering practice.
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 depth determination and are rarely used in civil engineering exploration.
Sonic method	The time of travel of sound waves reflected from the mud line beneath a body of water and a lower rock surface is computed by predetermining the velocity of sound in the various media.	Currently used in shallow underwater exploration to determine position of mud line and depth to hard stratum underlying mud. Method has been used in water depths greater than 100 ft with penetrations of 850 ft to bedrock but is presently in formative stage. Used most efficiently in water depths up to 50 ft with penetrations of additional 350 ft to bedrock. Possible future applications on land in determining ground water levels in pervious materials.

b. Accuracy. The greater the contrast in physical characteristics of subsurface strata, the sharper the measured response to applied impulse, and the more accurate the results. Consequently, the more successful applications are to profiles of soft soils underlain by rock.

c. Applications. These methods are suited to prospecting sites for dams, reservoirs, tunnels, highways, and large groups of structures.

(1) Seismic Refraction. Seismic refraction and electrical resistivity methods are those commonly used. For a review of techniques, see Hvorslev, *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes* (Bibliography).

(2) *Geophysical Procedures.* Geophysical procedures have been used to locate gravel deposits or other construction materials whose properties differ substantially from adjacent soils.

(3) *Seismic Soundings.* Seismic soundings, with sledge hammer blows to induce shock waves, determine rippability or need for blasting of dense and rocky subsurface materials.

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

2. LIMITATIONS. Geophysical surveys may be able to outline boundaries between strata, but soil properties are indicated only approximately by instrument response to applied impulse.

a. Sources of Error. Differences in degree of saturation, presence of mineral salts in ground water, or similarities of strata that effect transmission of source waves, may lead to vague or distorted conclusions.

b. Check Borings. Supplement a geophysical survey by borings to recover representative samples and to check stratification interpreted from the survey. Clearly illustrate correlation or lack of it between test borings and geophysical information in presenting subsurface data on contract drawings.

Section 5. TEST BORINGS

1. TYPES. See Table 2-3 for principal boring types and applications. For details of boring techniques and equipment, see Hvorslev, *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes* (Bibliography).

a. Selection of Boring Method. Choice of boring method depends on (a) the efficiency of the boring procedure in prospective materials, (b) ability to determine strata changes and material type, and (c) possible disturbance of materials to be sampled.

b. Specific Procedures.

(1) *Boring Without Sampling.* When only depth to rock or existence of cavities are to be determined, borings may be made without sampling and the utility of boring for sampling is not important.

(2) *Auger Borings.* Use these primarily for shallow exploration above ground water. Although materials recovered are disturbed, auger borings furnish continuous samples of soils encountered.

(3) *Wash Borings.* Use these borings for recovery of either disturbed dry samples or undisturbed samples. Ordinarily this is the type most suitable for locations with difficult access.

(4) *Rotary and Percussion Borings.* Use these borings for deep exploration or for penetration of hard soils or strata containing boulders and rock seams.

(5) *Rotary Core Drilling.* Use this procedure in bedrock to recover continuous core or to pass obstructions in overburden.

Section 6. SAMPLING DEVICES

1. THIN-WALL TUBE SAMPLERS. See Table 2-4 for principal thin-wall tube samplers used to obtain undisturbed samples.

a. Undisturbed Samples from Borings. These samples comply with the following criteria:

(1) They should contain no visible distortion of strata, or opening or softening of materials.

(2) Specific recovery ratio (length of undisturbed sample recovered divided by length of sampling push) should exceed 95 percent.

TABLE 2-3
Types of Test Borings

Boring method	Procedure utilized	Applicability
Displacement type . .	Repeatedly driving or pushing tube or spoon sampler into soil and withdrawing recovered materials. Changes indicated by examination of materials and resistance to driving or static force for penetration. No casing required.	Used in loose to medium compact sands above water table and soft to stiff cohesive soils. Economical where excessive caving does not occur. Limited to holes < 3" in diameter.
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.
Wash-type boring for undisturbed or dry samples.	Chopping, twisting, and jetting action of a light bit as circulating drilling fluid removes cuttings from hole. 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 over water, in swamps, on slopes, or within buildings.
Rotary drilling	Power rotation of drilling bit as circulating fluid removes cuttings from hole. Changes indicated by rate of progress, action of drilling tools, and examination of cuttings 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 rapid method of advancing bore hole.
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 usual higher cost. Sometimes used in combination with auger or wash borings for penetration of coarse gravel, boulders, and rock formations.
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.

(3) Area ratio of sampler (annular cross-sectional area of sampling tube divided by full area to OD of sampler) should be less than 15 percent.

b. Selection of Sampler. For undisturbed samples, use stationary piston samplers of the types in Table 2-4. The Swedish foil sampler is employed to recover long, continuous samples of soft soil. Other stationary piston samplers differ by various methods of locking the stationary piston during sampling. The choice between them is based on convenience or availability of equipment. The use of stationary pistons is less important in very stiff or hard cohesive soils; either a Shelby Tube or Pitcher sampler obtains essentially undisturbed samples.

2. THICK-WALL SPOON, AUGER, UNDERWATER, AND CORE BARREL SAMPLERS. For principal types, see Table 2-5.

a. Samples Recovered. These samples can be completely remolded, as provided by auger samplers, or merely disturbed as in the case of thickwall spoons. Samples are representative of soils in situ containing all constituents and suitable for index properties tests.

b. Core Barrel Samples. Generally, core barrel samples are taken in rock or hard cohesive soils (materials relatively insensitive to sampling disturbance). The suitability of cores for structural properties tests depends on the quality of individual samples. Specify double-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 sections with low recovery to estimate condition of missing sections and reasons for low recovery.

c. Wash Samples. Wash samples are taken from wash water circulated from boreholes and should not be relied on for identification of subsoils.

Section 7. SOUNDING AND PROBING DEVICES

1. EQUIPMENT. Sounding or probing consists of forcing a rod, a rod encased in a pipe sleeve, or a probe with pressure measuring device into subsoils to determine resistance to penetration or withdrawal.

a. Procedures. Methods comprise both dynamic and static resistance procedures. For details of equipment see Hvorslev, *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes* (Bibliography).

b. Penetration Resistance. Variation in penetration resistance indicates strata changes. For certain devices, resistance has been correlated with physical properties.

TABLE 2-4
Thin-Wall Tube Samplers with Medium to Low Area Ratio

Name	Procedure and equipment	Applicability and comments
	Stationary piston-type samplers	
Stationary piston sampler with piston rods.	Thin-wall sample tube is advanced beyond piston, generally by pushing drill rod while piston is held stationary by a clamp to the piston rod extension which runs through the sampler head and inside the drill rod to the surface. Cone lock in sampler holds piston to drill rod and prevents downward motion of piston.	Satisfactory for undisturbed sampling in soft to medium clays and silts from cased and uncased holes. Less successful for sands containing little or no cohesive material. Sampler has no positive device to keep piston from moving up before sampling. Recovers samples 24 in. long in thin-wall tubes of 2 to $4\frac{1}{2}$ in. OD.
Hydraulically activated piston sampler.	Thin-wall sample tube is advanced by pumping water or drilling mud through drill rod that holds the piston in a stationary position, eliminating the need for a separate set of piston rod extensions. Hydraulic pressure is confined on top of sample tube head by the outer tube until it builds up sufficiently to push sample tube down beyond the piston. The drill rod, holding the piston, is maintained stationary either by the drill chuck or other separate device.	Two common types for sampling in cased or uncased holes are the Osterberg Sampler and the Greer and McClelland Sampler. Not possible to observe amount of partial penetration of sampling tubes. Partial samples are likely to be disturbed. Satisfactory for soft to stiff clays and silts. Less successful for sand containing little or no cohesive material. Recovers samples 24 in. to 48 in. long in thin-wall tubes 3 to 5 in. OD.

TABLE 2-4 (Continued)
Thin Wall Tube Samplers with Medium to Low Area Ratio

Name	Procedure and equipment	Applicability and comments
	Stationary piston-type samplers	
Hong piston sampler.	Thin-wall sample tube is advanced beyond the piston by driving or pushing the drill rod. Piston is held stationary by an outer barrel that surrounds sample tube and sits on a special casing drive shoe, eliminating the need for a separate set of piston extension rods. A ratchet system holds piston in place for full penetration or any amount of partial penetration of sampler.	For undisturbed sampling in cased holes only. Satisfactory for soft to stiff clays and silts. Less successful for sands containing little or no cohesive material. Elimination of piston rods combined with advancement of tube by ordinary methods are principal advantages. Recovers samples 25-3/4 in. long in tubes 3 in. OD.
Swedish foil sampler.	16 rolls of 7/16-in.-wide thin metal strips housed inside the sampler head above a thin, sharp cutting edge envelop the sample as it enters the tube, thus minimizing friction between sample and tube. Sampler is advanced by pushing or jetting. During penetration, piston is held fixed by a chain that passes along inside of sample tube to the surface and is fixed to a stationary frame. Sample tubes are added until sampling run is completed.	Used for taking long, continuous, undisturbed samples in cohesive soils free of gravel, sand layers, or excessive shells, etc., which may break or rupture the foils and samples. Theoretically can recover samples 40 ft or more in length. Inside skin friction on sample is reduced to a minimum. Requires experienced operator. Sample tubes are 1/4 in. thick, 8 ft long sections of pipe, 2-3/4 in. ID.
	Nonpiston-type samplers	
Thin-wall Shelby tube sampler.	Thin-wall sampler is pushed or driven into the soil. Sampler head contains ball check to prevent water pressure from forcing sample from tube during withdrawal. Sampler accommodates tubes 2 to 4½ in. OD.	Used in soft to hard clays and silts or silty and clayey sands. Utilized for undisturbed samples of hard cohesive soils when driving is necessary to advance tube. Recovers samples 1-7/8 to 4-3/8 in. OD 24 in. long.
Pitcher sampler.	Thin-wall tube is forced into soil while core barrel outside tube reams out hole. Tube leads bit of core barrel an amount depending on consistency of soil. Tube prevented from rotating by ball bearing connection between tube and outer rotating barrel.	Sampler can be used in medium to soft clays and silts, but it is best adapted to hard clays and soft rock which are difficult or impossible to sample by other thin wall samplers. Recovers samples 3 to 5 ft long with 2½, 3, 4, and 5-7/8 in. OD.

(1) *Dynamic Resistance.* Dynamic resistance is measured by the number of blows of a particular weight dropped a specific height necessary for penetration of a sounder.

(2) *Static Resistance.* Variations in static resistance of a rod pushed or jacked into soil may be determined with greater accuracy than the dynamic resistance. Static values are easier to correlate with structural properties.

2. **UTILIZATION.** Unless penetration resistance is correlated with structural properties, soundings shall be relied on only to distinguish boundaries between strata.

a. Standard Penetration Test. The standard penetration test is the ordinary sounding procedure to determine compactness or hardness in situ; see Chapter 4.

b. Static Resistance Devices. Usually, static resistance devices are reserved for special exploration and require correlation of resistance readings with conventional test properties of subsoils.

TABLE 2-5
Samplers of Drive, Auger, Underwater, and Core Barrel Types

Type	Equipment and procedure utilized	Applicability
Drive samplers with high area ratio:		
Split spoon sampler.	Thick-wall, split barrel tube is driven or pushed into soil. Ball check in sampler head prevents water pressure from forcing sample out of tube. Split barrel opened in field for examination of sample. Sampler OD from 2 to 4½ in. Obtains samples 18 to 24 in. long, 1½ to 4 in. OD.	Used to obtain dry samples in practically all soils. Not adapted to coarse gravel and rock. The 2-in.-OD, 1½-in.-ID spoon is used for standard penetration resistance determination at each sampling depth. Numerous variations of this basic type are available for dry sampling.
Retractable plug sampler.	Sampler with inside plug is hand driven into the soil to depth of sampling desired. Plug then retracted and sampler driven into soil which enters a series of brass liners. Sampler removed from soil by jacking. Sampler OD 1.4 and 3 in. Obtain samples 42 and 60 in. long, 0.9 and 1.9 in. OD, respectively.	Portable device suitable for sampling clay, silt, and fine sands in inaccessible locations. Not appropriate for coarse grained sands. Equipment is simple, rugged, and requires little experience to use.
Split liner piston-type sampler.	Thin-wall stationary piston sampler with inside liner split longitudinally is advanced by driving or pushing. Liner removed after sampling and sample examined by separating liner halves. Sampler OD 2-1/8 in. Obtains samples 24 in. long, 1-7/8 in. in OD.	Suitable particularly for obtaining samples for field inspection or photographs. Utilized in stratified or varved sands with organic or clayey lenses. Other sampler types with relatively thick-wall split barrel and transversely split liner are utilized for a wide range of soil type. Samples obtained are moderately disturbed.
Auger samplers:		
Common auger samplers.	Samplers designed to completely remove soil from the hole. Advanced by hand or power rotation. Soil is completely churned upon removal. Some common types are helical or worm auger, Iwan augers, spoon, Vicksburg hinge auger, barrel auger, Buda continuous auger, disk auger, and large diameter bucket auger.	Used for continuous identification of materials in the profile and where tube samples are not required. Ordinarily used for shallow explorations above water table but can be taken to depths of about 50 ft.
Double-tube auger.	Open spiral sampler containing thin-wall liner is rotated into soil. Liner receives sample and open top cylinder retains material displaced by spiral. No water used. Sampler OD 1-1/4 and 2-1/4 in. Obtains samples 46 in. long.	Used for taking tube samples at relatively shallow depths. May be advanced by hand in soft to medium clays and loose to medium compact sandy soils.
Underwater samplers:		
Free fall gravity coring tube.	10- to 15-ft-long tube containing thin-wall liner is released at fixed height above ocean floor by utilizing pilot weight and lever mechanism. Sampler equipped with lead weights and stabilizing fins. Sampler OD 2-1/4 in. Obtains samples 1.9 in. OD.	Used for ocean bottom sampling. Successful in medium to stiff clays and in sand and gravel with sizes up to 1½ in. at depths exceeding 13,000 ft.
Piggot coring tube.	10-ft-long tube containing thin-wall liner is shot into ocean bottom by explosive charge that is triggered when sampler makes contact with the bottom. Sampler OD 2½ in. Obtains samples 1-7/8 in. in OD.	Successful for sampling stiff to hard ocean bottom soils in water depths exceeding 20,000 ft.
Harpoon-type sampler.	Thick-wall, split barrel tube with stabilizing fins is dropped through water under own weight to sample lower sediments. Sampler OD 2 in. Obtains samples 6 ft long, 1½ in. OD.	Developed by USCE for sampling river or harbor bottom muds and silts.

TABLE 2-5 (Continued)
Samplers of Drive, Auger, Underwater, and Core Barrel Types

Type	Equipment and procedure utilized	Applicability
Core barrel samplers:		
Denison sampler.	Similar to double-tube core barrel except cutting edge of inner barrel may be extended 3 in. beyond the outer coring bit by exchanging bits.	Adaptable to sampling hard clays, cemented coarse grained soils, hardpan, weathered, or soft rock. Sampler used in conjunction with rock coring when overburden consists of above materials which are difficult to sample by other methods.
Single-tube core barrel.	Tube with coring bit is rotated down into the rock, receiving core, while circulating water removes cuttings. Barrel OD from 1½ to 2-15/16 in. Obtains cores up to 10 ft long, from 7/8 to 2-1/8 in. OD.	Used primarily in sound rock not vulnerable to erosion, slaking, or fracturing.
Double-tube core barrel (swivel type).	Tube is enclosed within and attached to a core barrel by means of a swivel. As outer barrel rotates, inner tube remains stationary and receives core. Cuttings removed by circulating water between outer and inner barrels. Available with OD from 2 to 18 in.	Used primarily in nonuniform, fissured, friable, and soft rock. Obtains samples up to 20 ft long, 1-1/4 to 15-5/8 in. in OD.
Shot core barrel . .	Chilled steel shot is fed to rotating soft steel bit through drill rods and single barrel. Bit and rock are worn away as cuttings are washed above barrel by circulating water. Cuttings deposited in sludge barrel or calyx attachment. Available with OD 2 to 18 in. Obtains cores up to 20 ft long, 1-1/4 to 15-5/8 in. in OD.	Method for obtaining large diameter cores and drilling accessible boreholes in rock for exploration of dam sites. Used in medium hard and uniform rock. Shot often becomes embedded in soft rock and lost in seamy rock.

c. **Field Expedients.** On certain projects, expedient materials (rails or rods with detachable cone points) may be driven to determine stratification. Make typical soundings adjacent to borings to correlate penetration resistance with soil type.

Section 8. TEST PITS AND TRENCHES

1. **TYPES.** Test pits to facilitate examination or sampling of soils in situ range from shallow manual or machine excavations to deep, sheeted, and braced pits. For procedures in making test pits and hand samples, see Hvorslev, *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes* (Bibliography).

a. **Hand-cut Samples.** If properly taken, handcut samples of materials exposed in test pits may be of the highest quality. Certain materials such as loose sands, highly sensitive cohesive soils, or brittle and weathered rock may not furnish truly undisturbed samples in borings; hand-cut samples may be necessary.

b. **Machine Excavation.** Large diameter, rotary bucket augers are used to drill caisson holes that may be inspected and sampled directly. Pits or trenches made with backhoe, bulldozer, or clamshell bucket frequently are inexpensive if such equipment is available. Use mechanically excavated pits or trenches in lieu of equal expense in borings, where shallow conditions are essential in design. Determine locations of excavations and clearly identify and record them for reference during construction. Locate test pits so as not to disturb bearing materials at intended positions of shallow foundations.

Section 9. REQUIREMENTS FOR EXPLORATION PROGRAM

1. LAYOUT OF TEST BORINGS. Rules for preliminary and final borings are presented herein as general guides to planning and do not cover all the details necessary for specific sites or structures. Requirements for spacing of test borings are given below and in Table 2-6.

a. Preliminary Borings. For large sites, locate preliminary borings to furnish an overall subsoil survey rather than to follow a rigid geometric pattern.

b. Final Borings. Arrange final borings so geological sections may be determined at the most useful orientations. Borings in slide areas should establish geological sections necessary for stability analyses. To ensure that exploration is adequate in its final form, evaluate boring logs as received to develop a picture of subsoil conditions. Plan final borings for a certain sequence and make exploration contracts open ended so intermediate borings may be added in areas that prove to be critical.

c. Spacing Requirements. For additional information of spacing requirements, see also Table 2-6.

(1) Uniform Conditions. On large sites where subsurface conditions are relatively uniform, space preliminary borings 100 to 500 feet apart. Spacing is decreased in detailed exploration by intermediate borings as required to define variations in subsoil profile. Final spacing of 25 feet usually suffices for even erratic conditions.

TABLE 2-6
Requirements for Boring Layout

Areas for investigation	Boring layout
New site of wide extent	Space preliminary borings 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 warehouse building of large area.	Minimum of four borings at corners plus intermediate borings at interior foundations sufficient to define 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 dry sample 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 100 ft adding intermediate borings at critical locations, such as deep pumpwell, gate seat, tunnel, or culverts.
Long bulkhead or wharf wall	Preliminary borings on line of wall at 400 ft spacing. Add intermediate borings to decrease spacing to 100 or 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 establish geological section for analysis. Number of geological sections depends on extent of stability problem. For an active slide, place at least one boring upslope of sliding area.
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 and critical spots in abutment.
Highways and airfields	See NAVFAC DM-5 and NAVFAC DM-21 for general requirements for highways and airfields. For slope stability, deep cuts, and high embankments, see layout recommended above.

(2) *Cavities and Fractures.* Where factors such as cavities in limestone or fractures and joint zones in bedrock are being investigated, wash or rotary borings (without sample recovery) or soundings and probings are spaced as close as 10 feet center-to-center.

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

(4) *Subsurface Irregularities.* Inclined borings are required in special cases when surface obstructions prevent use of vertical holes, or subsurface irregularities such as buried channels, cavities, or fault zones are to be investigated.

2. DEPTHS OF TEST BORINGS. For general rules for boring depths, see Table 2-7. Required depths depend to some extent on sizes and types of proposed structures. They are controlled to a greater degree by the characteristics and sequence of subsurface strata.

a. Types of Strata. The depth of borings depends on the type of underlying strata.

(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.

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

TABLE 2-7
Requirements for Boring Depths

Areas for 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 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 1½ 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 ground water in stable materials, depth of 4 to 8 ft below base may suffice. Where base is below ground water, determine extent of pervious strata below base.
High embankments	Extend to depth between ½ and 1-1/4 times horizontal length of side slope in relatively homogeneous foundation. Where deep or irregular soft strata are encountered, borings should reach hard materials.
Dams and water retention structures . .	Extend to depth of ½ base width of earth dams or 1 to 1½ 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.
Highways and airfields	Extend auger borings to 6 ft below top of pavement in cuts, 6 ft below existing ground in shallow fills. For high embankments or deep cuts, follow criteria given above.
Airfields	Extend auger borings to 10 ft below top of pavement in cuts or 10 ft below existing ground in shallow fills.

(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 strata cannot affect stability or settlement.

(4) Bedrock Surface. If bedrock surface is to be determined and character and general location of rock are known, extend borings 5 feet into sound, unweathered rock. Where character of rock is unknown, or where boulders or irregularly weathered material overlie bedrock, core 10 feet into sound rock and include 20 feet of coring in one or two selected borings. In cavitated limestone, extend borings through strata suspected of containing solution channels.

b. Check Borings. During final exploration, at least one boring should extend well below the zone involved in the apparent stability, settlement, or seepage problem to make sure no unusual conditions exist at greater depth.

c. Securing Borings. Borings made in foundation areas that eventually will be excavated below ground water, or where artesian pressures are encountered, must be plugged or grouted unless used for continuing water-level observations. In boreholes for ground water observations, place casing in tight contact with walls of hole.

3. REQUIREMENTS FOR SAMPLING PROGRAM. The number, type, and distribution of boring samples depend on strata arrangement and sample usage.

a. Representative Dry Samples. Representative dry samples are generally obtained at vertical intervals no less than 5 feet center-to-center of sample location and at every change in strata.

b. Undisturbed Samples. Precede recovery of undisturbed samples by dry sample borings to determine thickness and extent of critical strata. Number and spacing of undisturbed samples depend entirely on related design problems and necessary testing program.

(1) Spacing of Samples. Distribution of undisturbed samples may range from only one or two in a boring to practically continuous undisturbed samples in a critical stratum.

(2) Continuous Sampling. Consider using the Swedish foil sampler if continuous sampling is necessary for sand drain design or elaborate settlement or stability analyses.

4. OBTAINING UNDISTURBED SAMPLES. Obtain undisturbed samples in cohesive soil strata, so that at least one representative sample is taken from each 10 feet of cohesive soil in each boring. Include appropriate requirements in specifications for contract borings. The procedures for obtaining undisturbed samples are:

a. Caving. Use casing or viscous drilling fluid to advance borehole if there is danger of caving. Where casing is not necessary, drilling fluid is preferred.

b. Aboveground Water Table. When sampling aboveground water table, maintain borehole dry whenever possible.

c. Belowground Water Table. When sampling belowground water table, maintain borehole full of water or drilling fluid during cleanout, during sampling and sample withdrawal, and while removing cleanout tools. If necessary, this should be accomplished by positive inflow at ground surface. At depths where continuous samples are required, casing should remain full of drilling fluid for the entire drilling and sampling operation.

d. 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.

5. EQUIPMENT FOR OBTAINING UNDISTURBED SAMPLES. The equipment necessary for obtaining undisturbed samples is listed below.

a. Sampling Devices. The appropriate sampler listed in Table 2-4 should be used.

b. Sampling Tubes. The requirements for sampling tubes depend on several factors.

(1) Composition. Although brass or hard aluminum tubes are preferred, steel tubes may be used if they are coated on both the inside and the outside with a hard, smooth, noncorrosive lacquer.

(2) Interior. Tubes should be seamless with clean, smooth interior, free of protrusions.

(3) Cutting Edge. Tubes should have machine prepared sharp cutting edge with outside taper not exceeding 10°. Do not permit rounded or blunt cutting edges.

(4) Inside Clearance. Provide tube with inside clearance between 0.5 percent (for partially saturated or soft clays and silts) to 3.0 percent (for hard, saturated swelling clays). Inside clearance = $\frac{D_e - D_e}{D_e}$, where D_s is ID of tube above cutting edge. Cutting edge should be drawn in by machine to provide inside clearance.

c. Casing. Use casing in upper 10 feet of all borings and throughout boring in soils with cavities, or in soils where full head of drilling fluid cannot otherwise be maintained at all times. Use casing whenever required to obtain ground water observations at intermediate depths for extended periods, and in general whenever required for successful boring operations.

(1) Advancing Casing. Drive casing down in stages not exceeding 5 feet between cleanout. Do not advance casing by using it as a wash pipe. Unless necessary to maintain stability of borehole, do not drive the casing in advance of borehole. In cohesive soils, it is permissible to advance the hole as far as possible without using the casing, unless the casing is needed to perform sampling operations.

(2) Special Sampling Conditions. When sampling loose to medium compact, cohesionless soils and saturated silt, the casing should not be advanced the last 2 feet by drive weight, but may be pushed hydraulically or rotated in combination with an advance cleanout operation taking care not to disturb the intended top of sample. During continuous sampling operations, never drive the casing to sample depth of drive weight.

6. DETAILED PROCEDURES. The detailed procedures for cleaning the borehole and collecting and preserving samples are outlined below.

a. Cleaning Borehole. The following steps should be taken to clean the borehole.

(1) With or Without Casing. Cleaning of borehole with or without casing should not be done through open-end drill rod or sampling spoon.

(2) Jetting. Downward or sideward jetting is not permitted when cleaning below casing. Use any jet auger that deflects the flow of water or drilling fluid upward.

(3) Jet Bits. Cleanout with jet bits that direct the flow downward or sideward is permitted within the casing, but should not be done within 4 inches of intended top of sample. The last 4 inches is cleaned out with any jet auger that deflects water or drilling fluid upward.

(4) Sand Pump or Bailer. Do not use sand pump or bailer within 12 inches of intended top of sample.

(5) Viscous Drilling Fluid. Whether or not casing is present, always use viscous drilling fluid instead of clear water when sampling saturated cohesionless sands. Drilling fluid may be necessary when sampling other soil types to prevent sample loss, excess swell, and disturbance in the vicinity of the sample.

(6) Casing Tip. When the casing is extended to sample depth, all soil must be cleaned out to at least the casing tip and preferably about 4 inches below the tip. Where continuous samples are taken, allow for this 4 inches when determining final depth of casing before sampling.

(7) Coarse Material. Coarse washed material must be removed from borehole before sampling, and the hole should be cleaned so that soil at the intended top of the sample is as nearly undisturbed as possible.

(8) Sample Retrieval. Take the sample as soon as possible after cleaning the hole. Cleaning of the hole should not be attempted if sampling is to be delayed.

b. Sampling Operation. The following steps are to be followed when collecting the sample.

(1) Preparation. Sampler and tube must be properly cleaned with vents, valves, piston packing, etc., checked for proper placement and function.

(2) Lowering Tube. Lower sampler slowly and carefully to bottom of hole without dropping. When encountering water table while lowering the sampler, precautions must be taken with samplers containing piston rod extensions to prevent an upward rise of the piston.

(3) Securing Piston Rods. Provide piston extension rods with a positive locking device at ground surface, and securely lock piston rods before sampling.

(4) Penetration. Force the sample tube past the locked piston by uninterrupted hydraulic pushing. Do not rotate sample tube during downward movement.

(5) Length of Penetration. Length of sample penetration should never exceed net length of sampler. For sampling tubes of 2 inches ID, penetration should not exceed 10 times ID for cohesionless soils, or 15 times ID for cohesive soils.

(6) Withdrawal. After penetration, allow sampler to sit for at least 10 minutes before withdrawal. Then rotate sample tube 2 to 3 revolutions and withdraw slowly using moderate upward pull on drill rod, avoiding sudden acceleration, shock, or vibration.

(7) Tube Removal. After withdrawing the sampler from the hole, take care not to drop it on the ground. Remove the tube from the sampler head without disturbing the sample.

c. Sample Preservation. The procedures for sample preservation are as follows.

(1) Handling. Handle sample tubes with extreme care at all times after removal from borehole.

(2) Sealing. Before sealing, remove any disturbed material from the tube and clean tube walls to provide good contact for sealer wax. After waxing the ends of the tube, place snugly fitting metal caps at each end and tape them to the sample tube. Again, immerse the tube ends in wax. When there is an annular clearance between the sample and tube that cannot be completely sealed, remove the sample from the tube and wax the sample completely in a large container. If too great an inside tube clearance is suspected, obtain new tubes having a smaller clearance before further samples are taken.

(3) Identification. Mark sample tubes with boring number, sample number and depth, total drive, measured recovery of undisturbed soil before trimming, and description of soil type at the upper end of the tube.

(4) Protection. Protect sample from extreme heat and freezing after withdrawal from hole and during transportation.

(5) Packing. Pack sample tubes for shipment with excelsior or sawdust in sturdy boxes large enough to contain not more than six tubes. Mark boxes "Fragile," "Protect From Freezing and Extreme Heat."

(6) Sample Retention. Indefinite storage of samples is not warranted. They should normally be retained only until the construction contract is awarded.

CHAPTER 3. LABORATORY TESTS AND TEST PROPERTIES

Section 1. INTRODUCTION

1. **SCOPE.** This chapter covers laboratory test procedures, typical test properties, and the application of tests to design and construction. Symbols and terms relating to tests and soil properties conform, generally, to definitions given in ASTM D653.

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

Subject	*Source*
Tests for airfields	NAVFAC DM-21
Tests for highways	NAVFAC DM-5

3. **LABORATORY EQUIPMENT.** For lists of laboratory equipment required for performance of tests, see Lambe, *Soil Testing for Engineers* and other criteria sources given for test procedures.

4. **TEST SELECTION FOR DESIGN.** Standard or suggested test procedures, variations which may be required, and type and size of sample are included in Tables 3-1, 3-2, and 3-3; Table 3-4 contains soil properties determined from tests and their applications.

a. Index Properties Tests. Index properties are used to classify soils, to group soils in major strata, and to extrapolate results from a restricted number of structural properties tests to determine properties of other similar materials. Procedures for most index tests are standardized (Table 3-1). Either representative dry samples or undisturbed samples are utilized. Tests are assigned after review of boring data and visual identification of samples recovered.

b. Structural Properties Tests. These must be planned for particular design problems. Rigid standardization of test procedures is inappropriate (Table 3-2). 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 disturbance. 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 undisturbed samples is preferred to many mediocre tests on specimens selected at random.

c. Compacted Sample Tests. In prospecting for borrow materials, index tests or tests specifically for compacted samples 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. Select samples for test to represent the main soil groups and probable compacted condition.

TABLE 3-1
Requirements for Index Properties Test

Test	Reference for standard test procedure	Variations from standard test procedure	Type of sample for test[1]	Size or weight of sample for test[2]
Sample preparation. . . .	ASTM D421	None	Disturbed or undisturbed.	As required for subsequent tests.
Moisture content	ASTM D2216	None	Disturbed or undisturbed with unaltered natural moisture content.	As large as convenient.
Dry unit weight.	None	Determine total dry weight of a sample of measured total volume.	Undisturbed with unaltered natural volume.	As large as convenient.
Specific gravity:				
Material smaller than No. 4 sieve size.	ASTM D854	Volumetric flask preferable; vacuum preferable for de-airing.	Disturbed or undisturbed.	25 to 50 gm for fine grained soils; 150 gm for coarse grained soil.
Material larger than No. 4 sieve size.	ASTM C127	None	Disturbed or undisturbed.	500 gm.
Atterberg limits:				
Liquid limit	ASTM D423 (One-point method AASHO T89)[3]	Harvard liquid limit device and grooving tool acceptable;[3] open wire grooving tool acceptable.	Disturbed or undisturbed, fraction passing No. 40 sieve.	50 to 100 gm.
Plastic limit	ASTM D424	Ground glass plate preferable for rolling. Material for Atterberg limit tests should not be dried before use.	Disturbed or undisturbed, fraction passing No. 40 sieve.	15 to 20 gm.
Shrinkage limit	ASTM D427	In some cases a trimmed specimen of undisturbed material may be used rather than a remolded sample.	Disturbed or undisturbed.	30 gm..
Gradation:				
Sieve analysis	ASTM D422	Selection of sieves to be utilized may vary for samples of different gradation.	Disturbed or undisturbed, nonsegregated sample, fraction larger than No. 200 sieve size.	600 gm for finest grain soil; to 4,000 gm for coarse grained soils.
Hydrometer analysis . .	ASTM D422	Fraction of sample for hydrometer analysis may be that passing No. 200 sieve. Entire sample of fine grained soil may be used.	Disturbed or undisturbed, nonsegregated sample, fraction smaller than No. 10 sieve size.	65 gm for fine grained soil; 115 gm for sandy soil.

1. Disturbed or undisturbed indicates that the source sample may be of either type. ASTM standard test procedures given in *Standards, Part 4;* AASHO standard test procedures given in *Standards, Part III.*

2 Weights of samples for tests on air-dried basis.

3 Lambe, *Soil Testing for Engineers.* One-point method of liquid limit test must not be used for control of construction or for determining compliance with specification requirements.

TABLE 3-2
Requirements for Structural Properties Tests

Test	Reference for suggested test procedure	Variations from suggested test procedure	Size or weight of sample for test (undisturbed, remolded, or compacted)
Permeability:			
Constant head procedure.	(1)	For clean, coarse grained soils, the procedure of ASTM[2] is preferable.	Sample size depends on max grain size, 4 cm dia by 35 cm height for silt and fine sand.
Variable head procedure.	(1)	Generally applicable to fine grained soils.	Similar to constant head sample.
Constant head procedure for coarse grained soils.	ASTM[2]	Limited to soils containing less than 10% passing No. 200 sieve size.	Diameter varies from 3 in. for 3/8 in. max grain size, to 6 in. for 3/4 in. max grain size.
Capillary head . . .	(1)	Capillary head for certain fine grained soils may have to be determined indirectly.	200 to 250 gm dry weight.
Consolidation . . .	(1) ASTM[2]	To investigate secondary compression, individual loads may be maintained for more than 24 hr.	Diameter preferably 2½ in. or larger. Ratio of diameter to thickness of 3 or 4.
Direct shear.	(1) ASTM[2]	Limited to tests on cohesionless soils or to consolidated shear tests on fine grain 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.	(1)	Alternative procedure given in ASTM[2] (p. 321).	Similar to triaxial test samples.
Triaxial compression:			
Unconsolidated—undrained (Q or UU). Consolidated—undrained (Qc or UD). Consolidated—drained (S or D).	(1)	For measurement of pore water pressures during test, special additional equipment is required. In this case rate of shear must be no faster than certain limiting speeds.	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; 1.4 in. dia, 3.5 in. high.

[1] Lambe, *Soil Testing for Engineers*.
[2] ASTM, *Procedures for Testing Soils*.

d. 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 should be investigated 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 3-5). Ordinarily, determine moisture content for all the representative samples (disturbed or undisturbed) for classification and grouping of materials in principal strata.

TABLE 3-3
Requirements for Compacted Samples Tests

Test	Reference for standard test procedure[1]	Variations from standard test procedure	Size or weight of sample for test[2]
Moisture-density relations: Standard Proctor 5½-lb hammer, 12-in. drop.	ASTM D698	Preferable not to reuse samples for successive compaction determinations.	Each determination: Method A: 6 lb Method B: 14 lb Method C: 10 lb Method D: 22 lb
Modified Proctor 10-lb hammer, 18-in. drop.	ASTM D1557	Preferable not to reuse samples for successive compaction determinations.	Method A: 7 lb Method B: 16 lb Method C: 12 lb Method D: 25 lb
Harvard compaction apparatus . .	ASTM[2]	None	2 to 3 lb of material passing No. 4 sieve size for complete curve.
Maximum and minimum densities of cohesionless soils.	ASTM D2049	Alternative methods using vibrating tamper given in ASTM.[3]	Varies from 10 to 130 lb depending on max. grain size.
Moisture-penetration resistance relations.	ASTM D1558	None	As required for compaction. Methods A or B above.
California bearing ratio	USCE TM-5-852-6 Appendix III.	Compaction energy other than that for Modified Proctor may be utilized.	Each determination requires 15 to 25 lb depending on gradation.
Expansion pressure	AASHO T174	None	10 to 15 lb depending on gradation.
Permeability and compression characteristics.	U. S. Bureau of Reclamation E-12.	Testing procedures of Table 3-2 may be utilized.	15 lb of material passing No. 4 sieve size.

[1] For other sources of standard test procedures, see Table 3-1.
[2] Weight of samples for tests given on air-dried basis.
[3] *Procedure for Testing Soils* (ASTM).

a. **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 may be determined with sufficient accuracy for computation of loads by assuming saturation.

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

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.

a. **Grain-Size Parameters.** Gradation, in simplest terms, is D_{10} size (which is an approximate measure of the size of the void spaces in coarse grain soils) and the coefficient of uniformity D_u, which indicates the relative broadness or narrowness of gradation.

b. **Test Schedule.** 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.

TABLE 3-4
Soil Properties for Analysis and Design

Property	Symbol	Units[1]	How obtained	Direct applications
Volume-weight characteristics:[2]				
Moisture content	w	D	Directly from test	Classification and in volume-weight relations.
Unit weights	γ	FL^{-3}	Directly from test or from volume-weight relations.	Classification and for pressure computations.
Porosity	n	D	Computed from volume-weight relations.	Parameters used to represent relative volume of solids in total volume of soil.
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	Classification.
Plasticity index	PI	D	LL-PL	Classification and properties correlation.
Shrinkage limit	SL	D	Directly from test	Classification and computation of swell.
Shrinkage index	SI	D	PL-SL	
Activity	A_c	D	$\frac{PI}{\%\ \text{"clay size"}}$	Identification of clay mineral.
Liquidity index	LI	D	$\frac{w - PL}{PI}$	Estimating degree of preconsolidation.
Gradation characteristics:				
Effective diameter	D_{10}	L	From grain-size curve	Classification, estimating permeability and unit weight, filter design, grout selection, and evaluating potential frost heave.
Percent grain size	D_{30}, D_{60}, D_{85}	L	From grain-size curve	
Coefficient of uniformity. . .	C_u	D	$\frac{D_{60}}{D_{10}}$	
Coefficient of curvature . . .	C_c	D	$\frac{(D_{30})^2}{(D_{10}) \times (D_{60})}$	
Clay size fraction	. . . . . .	D	From grain-size curve, % finer than 0.002 mm.	Classification.
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 arith. –e vs p curve.	Computation of ultimate settlement or swell in consolidation analysis.
Coefficient of volume compressibility.	m_v	L^2F^{-1}	$\frac{a_v}{1 + e}$	
Compression index	C_c	D	Determined from semilog e vs p curve.	
Recompression index	C_r	D		
Swelling index	C_s	D		
Coefficient of secondary compression.	C_α	D	Determined from semilog time-consolidation curve.	
Coefficient of consolidation.	c_v	L^2T^{-1}		Computation of time rate of settlement.
Preconsolidation pressure. .	P_c	FL^{-2}	Estimated from semilog e vs p curve.	Consolidation analysis.

TABLE 3-4 (Continued)
Soil Properties for Analysis and Design

Property	Symbol	Units[1]	How obtained	Direct applications
Shear strength characteristics:				
Angle of internal friction . .	ϕ	D	Determined from Mohr envelope for total normal stress.	Analysis of stability and load carrying capacity of foundations.
Cohesion intercept	c	FL^{-2}		
Angle of internal friction . .	ϕ'	D	Determined from Mohr envelope for effective normal stress.	
Cohesion intercept	c'	FL^{-2}		
Unconfined compressive strength.	q_u	FL^{-2}	Directly from test.	
Shear strength	s	FL^{-2}		
Sensitivity	S_t	D	$\frac{q_u\text{ (undisturbed)}}{q_u\text{ (remolded)}}$	Estimating effect of disturbance.
Modulus of elasticity	E_s	FL^{-2}	Determined from stress-strain curve.	Computation of elastic settlement or rebound.
Characteristics of compacted samples:				
Maximum dry unit weight. . .	γ_{max}	FL^{-3}	Determined from moisture-density curve.	Compaction control and computation of weights and forces in stability analysis.
Optimum moisture content . .	OMC	D		
Needle penetration resistance.	P_r	FL^{-2}	Directly from test	Moisture control in compaction.
Relative density	D_d	D	Determined from results of max and min density tests.	Compaction control.
California bearing ratio . . .	CBR	D	Directly from test	Pavement design.

[1] Units: F = force or weight; L = length; T = time; D = dimensionless.

[2] For complete list of volume-weight relationships, see Table 3-5.

3. **ATTERBERG LIMITS.** For classification of fine grained soils by Atterberg limits, see Figure 3-1. In addition to their use in soil classification, Atterberg limits also are indicators of structural properties, as shown in the correlations in this chapter. Limit tests should be performed discriminately, and should be reserved for representative samples selected after evaluating subsoil pattern. Determine 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 contents.

Section 3. PERMEABILITY TESTS

1. **UTILIZATION.** Permeability coefficient is used to compute the quantity and rate of water flow through soils in drainage and seepage analysis.

a. Applications. Laboratory tests are appropriate for compacted materials in dams, filters, or drainage structures.

(1) Fine Grained Soils. Permeability of fine grained soils generally is computed from consolidation test data or by direct measurement on consolidation or triaxial shear specimens.

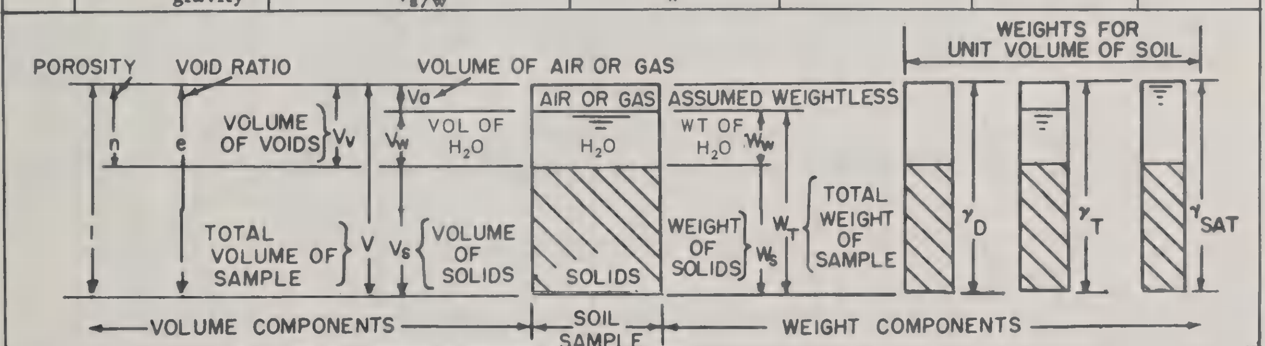
(2) Sand Drain Design. Sand drain design may require complete permeability data for soils to be stabilized, including determination of permeabilities for both vertical and horizontal direction.

(3) Field Permeability Tests. The secondary structure of in situ soils, stratification, and cracks has a great influence on the permeability. Results of laboratory tests should be interpreted with this in mind, and field permeability tests (Chapter 4) should be performed where warranted.

b. Typical Values. Coefficient of permeability is the property most sensitive to sample disturbance, and shows the widest range of variation of ordinary structural characteristics. See Terzaghi and Peck,

TABLE 3-5
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	[a] $\frac{W_s}{G\gamma_w}$		$V-(V_a+V_w)$	$V(1-n)$	$\frac{V}{(1+e)}$	$\frac{V_v}{e}$
	V_w	volume of water	$\frac{W_w}{\gamma_w{}^*}$		$V_v - V_a$	SV_v	$\frac{SV_e}{(1+e)}$	$SV_s\,e$
	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	$\frac{V_v}{V}$		$\frac{1-V_s}{V}$	$1-\frac{W_s}{GV\gamma_w}$	$\frac{e}{1+e}$	
	e	void ratio	$\frac{V_v}{V_s}$		$\frac{V}{V_s - 1}$	$\frac{GV\gamma_w}{W_s} - 1$	$\frac{W_w G}{W_s S}$	$\frac{n}{1-n}$ | $\frac{wG}{S}$
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	$\frac{W_s}{V_s\gamma_w}$		$\frac{Se}{w}$			



[a]–γ_w is unit weight of water, which equals 62.4 pcf for fresh water and 64 pcf for sea water (1.00 and 1.025 gm/cc). (Where noted with * the actual unit weight of water surrounding the soil is used.) In other cases use 62.4 pcf.

Values of w and s are used as decimal numbers.

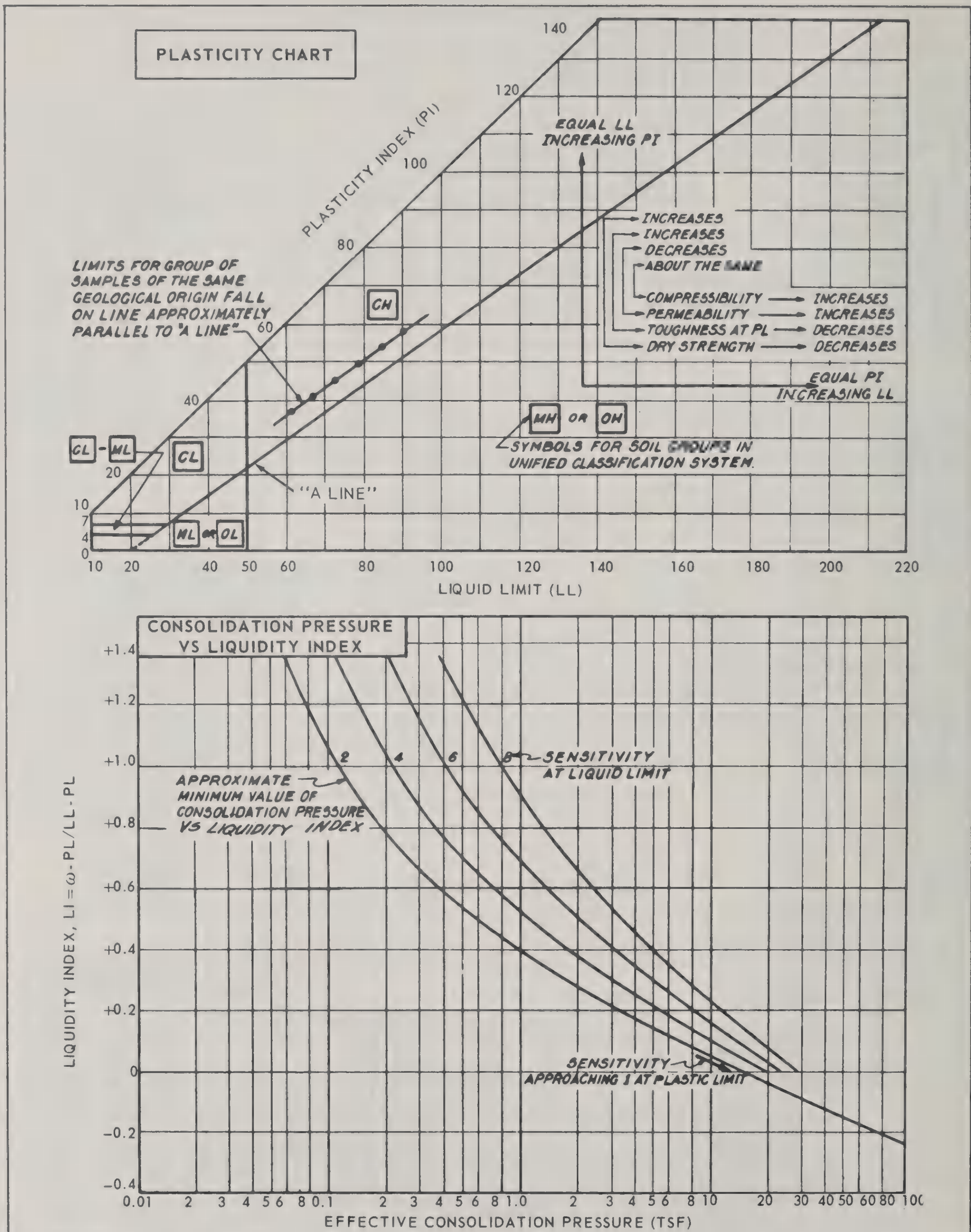


FIGURE 3-1
Utilization of Atterberg Plasticity Limits

Soil Mechanics in Engineering Practice, for correlations of permeability with soil type. Permeability to clean, coarse grain samples is related to D_{10} size (Figure 3-2). Standard deviation of this correlation is about $\pm$ log 3; that is, two-thirds of the random values fall between three times the average value and one-third of the average value shown.

Section 4. CONSOLIDATION TESTS

1. UTILIZATION. One-dimensional consolidation tests with complete lateral confinement determine total compression of fine grained soil under applied load and time rate of compression caused by gradual volume decrease that accompanies the squeezing of pore water from the soil. See Figure 3-3 for test relationships.

a. Test Schedule. Consolidation tests require undisturbed samples of highest quality. Select samples representative of principal compressible strata. Determination of consolidated characteristics of a stratum requires from two or three to about eight tests, depending on the complexity of conditions.

b. Loading Sequence. Ordinarily, apply loads starting at 1/4 tsf and increase them geometrically to 1/2, 1, 2, 4, 8, etc., tsf. For soils with pronounced swelling tendency, it may be necessary to commence loading at 1/2 tsf or higher 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.

2. PRECONSOLIDATION PRESSURE. The pressure value forms the boundary between recompression and virgin compression ranges and is an estimate of the maximum normal stress to which the material in situ has been subjected by a previous loading.

a. Determination. Estimate preconsolidation stress from semilogarithmic pressure-void ratio curve. Use the procedure given in Terzaghi and Peck, *Soil Mechanics in Engineering Practice.* Maximum test pressures should exceed preconsolidation by an amount sufficient to define the slope of virgin compression. Generally, this requires application of three load increments exceeding the preconsolidation value.

b. Approximate Values. Preconsolidation stress is related to liquidity index. See bottom panel of Figure 3-1 (standard deviation $\pm$ log 2). 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.

3. VIRGIN COMPRESSION. Virgin compression is 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 recompression and virgin compression ranges. The semilogarithmic, straight-line slope for virgin compression is expressed by the compression index C_c.

b. Approximate Values. The compression index C_c of silts and clays has been correlated with the natural water content and the liquid limit. Approximate correlations from Terzaghi and Peck, and from Nishida are, respectively:

$$C_c = 0.009\,(LL - 10). \tag{3-1}$$

$$C_c = 0.0054\,(2.6w - 35). \tag{3-2}$$

where w = natural water content, in percent.

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).

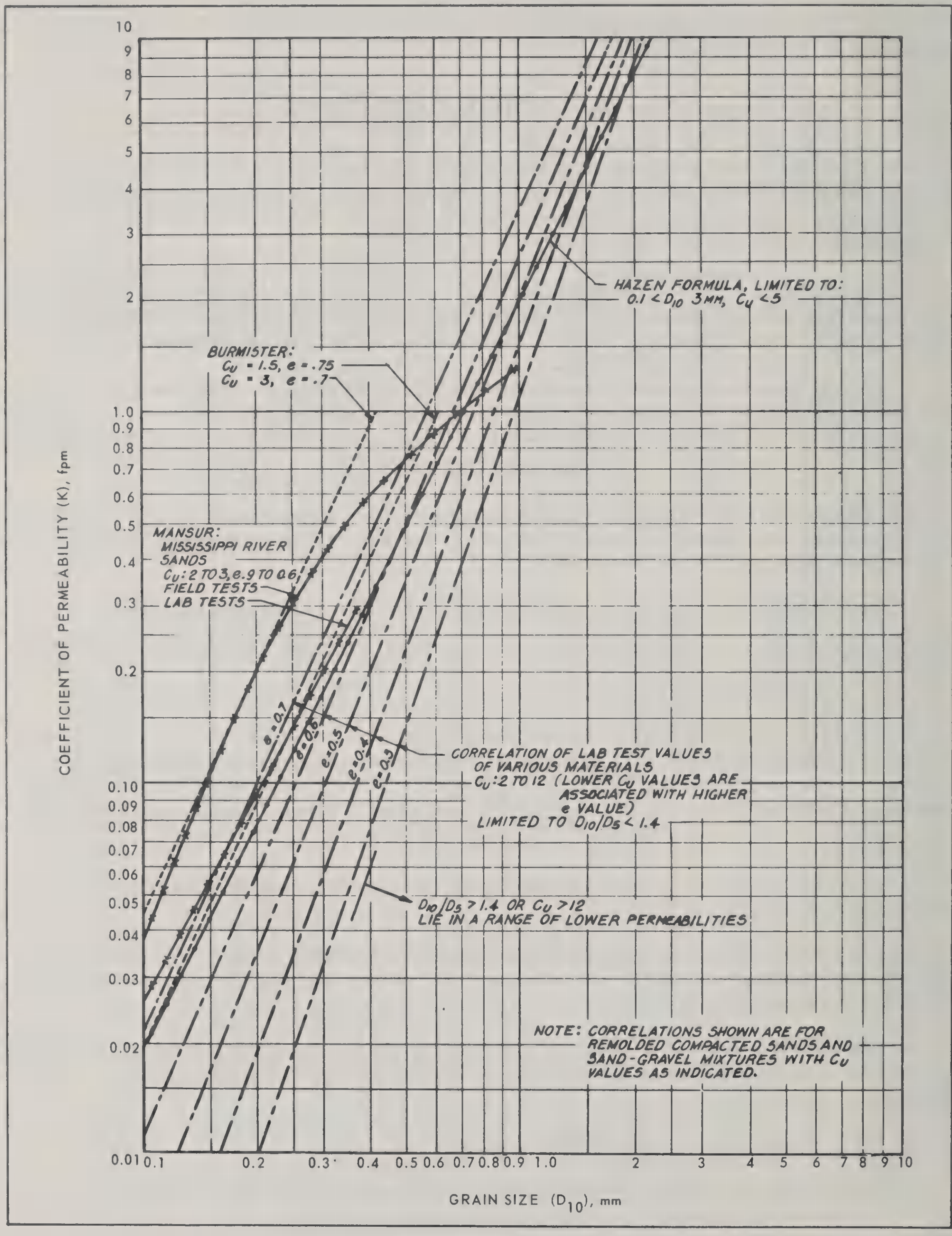


FIGURE 3-2
Permeability of Sands and Sand-Gravel Mixtures

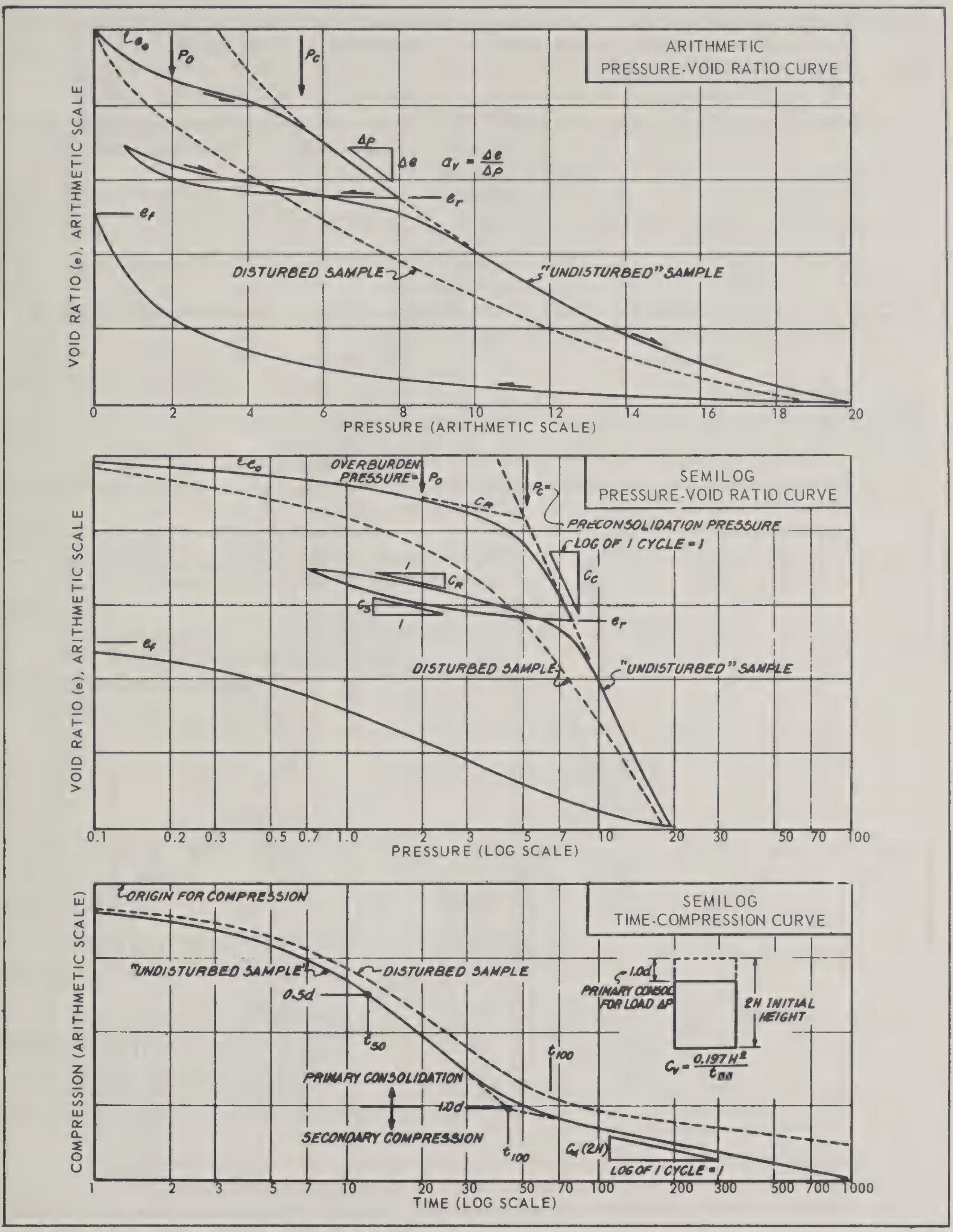


FIGURE 3-3
Consolidation Test Relationships

4. 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. The swelling index is generally one-fifth to one-tenth of the compression index. For typical values of C_s, see Figure 3-4 (standard deviation ± 50 percent).

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.

5. 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 specific load increment (bottom panel of Figure 3-3). Correct the origin for compression for the effect of air or gas in void spaces by the procedure given in Lambe, *Soil Testing for Engineers.*

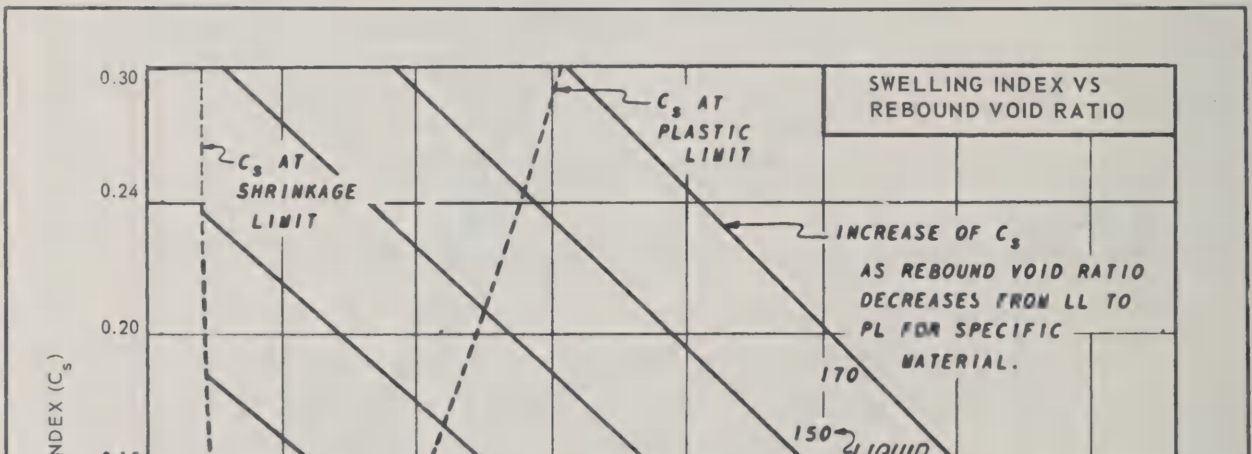


FIGURE 3-4
Approximate Correlations for Swelling Index of Silts and Clays

b. Approximate Values. For virgin compression of undisturbed samples, the standard deviation of c_v correlation equals ± log 2. The lower limit for recompression and upper limit for remolded samples are boundaries of zones within which a wide scatter of values is observed (top panel of Figure 3-5).

6. SECONDARY COMPRESSION. After completion of primary consolidation under a specific load, the semilogarithmic time-compression curve continues approximately as a straight line, which is the range of secondary compression (Figure 3-3).

a. Organic Materials. In organic materials, secondary compression is particularly important and may dominate the time-compression curve, accounting for more than one-half of the total compression.

b. Approximate Values. The coefficient of secondary compression C_α is the ratio of decrease in sample height to initial sample height for one cycle of time on log scale. See the bottom panel of Figure 3-5 for typical values (standard deviation ± 50 percent).

7. SAMPLE DISTURBANCE. A comparison of consolidation tests on undisturbed and moderately disturbed samples is shown in Figure 3-3.

a. Influence on Test Values. The influence of sample disturbance on test values is listed below.

(1) Void Ratio. Sample disturbance lowers the void ratio reached under any applied pressure and eliminates a distinct break in pressure-void ratio curve at the preconsolidation stress.

(2) Preconsolidation Pressure. Sample disturbance lowers preconsolidation pressure interpreted and compression index value.

(3) Recompression and Swelling. Sample disturbance increases recompression and swelling indices.

(4) 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 eliminated in badly disturbed samples.

(5) Coefficient of Secondary Compression. Sample disturbance decreases coefficient of secondary compression in virgin compression 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. LABORATORY SHEAR TESTS. Many types and variations of shear tests have been developed. The following are the most widely used.

a. Direct Shear Test. A thin soil sample is confined between parallel blocks and sheared along a predetermined plane.

b. Unconfined Compression Test. A cylindrical sample is loaded in compression. Failure occurs along diagonal planes where the greatest ratio of shear stress to shear strength occurs.

c. Triaxial Shear Test. Similar to the unconfined compression test (except that the sample is confined laterally by a membrane and fluid pressure), the triaxial test (Figure 3-6) is the most sophisticated shear test and permits testing under a variety of confinement, consolidation, and drainage conditions.

3. TEST SELECTION. In determining the type of test to be employed, the following considerations shall be observed.

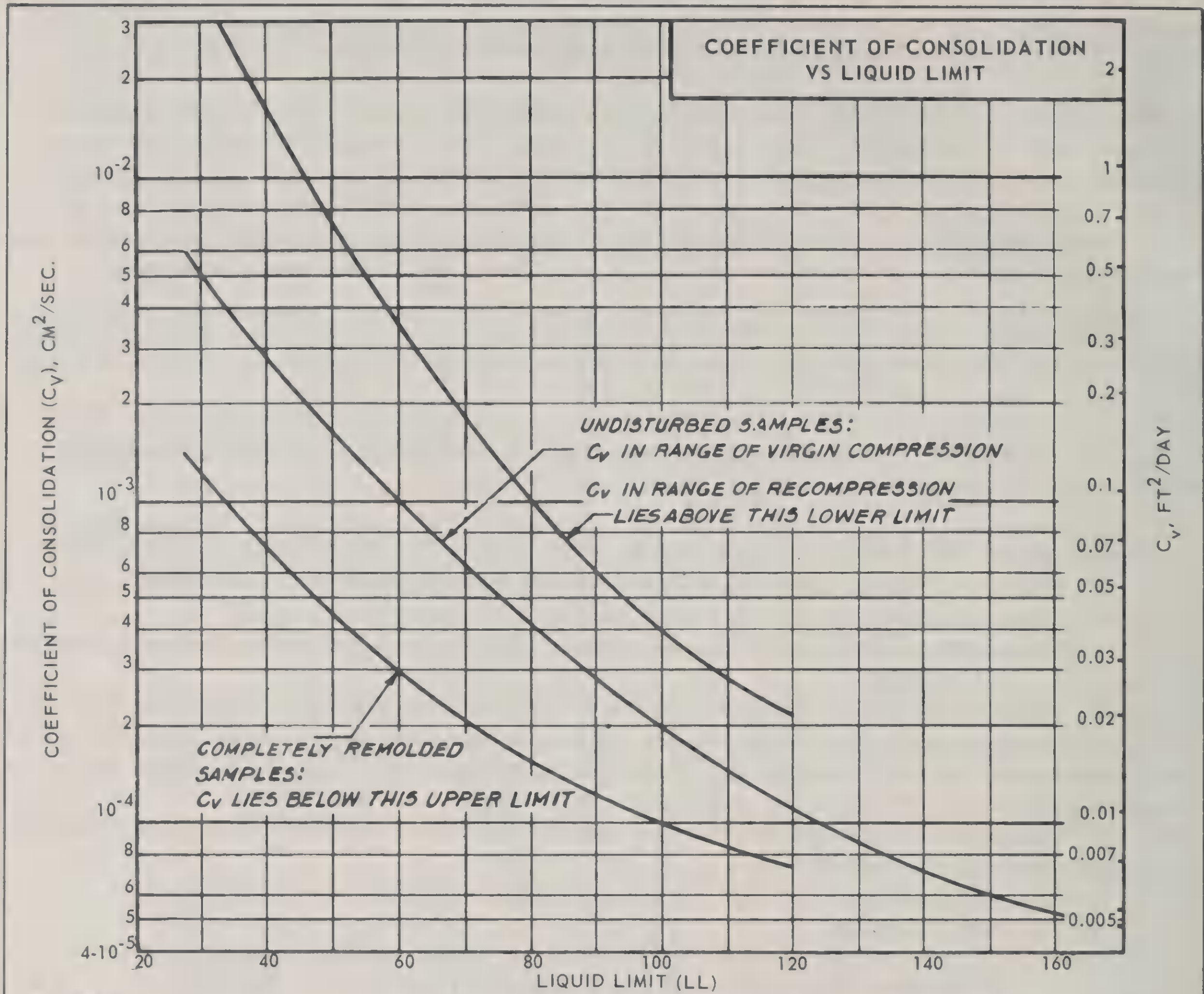


FIGURE 3-5
Approximate Correlations for Consolidation Characteristics of Silts and Clays

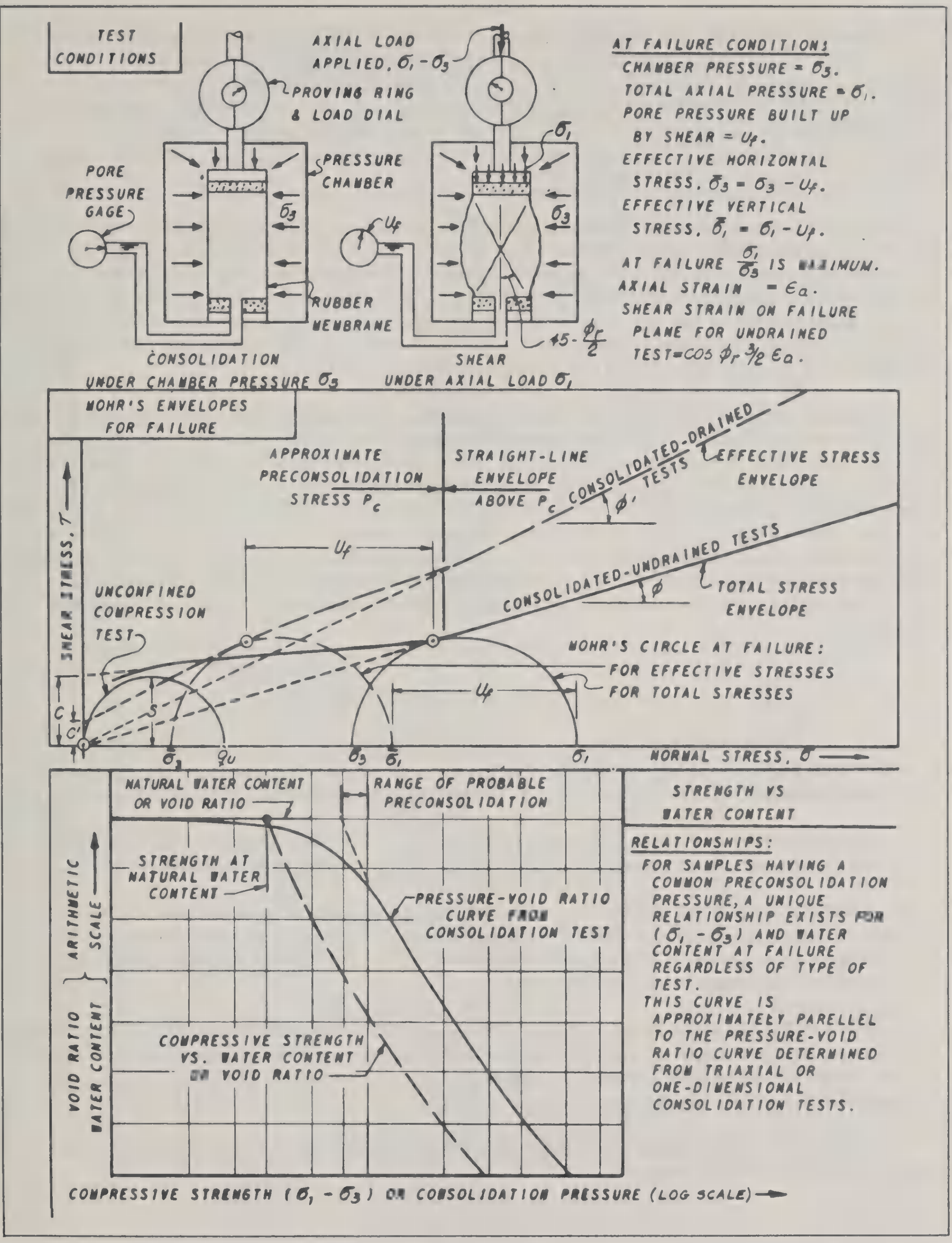


FIGURE 3-6
Triaxial Shear Test Relationships

Metz Reference Room
University of Illinois
B106 NCEL
208 N. Romine Street
Urbana, Illinois 61801

a. Clean Sands and Gravels. Undisturbed samples are not possible, and sophisticated shear tests are not warranted. For most foundation problems, the angle of internal friction can be satisfactorily approximated by correlation with penetration resistance, relative density, and soil classification (Table 1-3 and Figure 3-7). Further confirmation of the potential range of the angle of internal friction can be obtained from direct shear tests on the sample at its densest and loosest possible states in the laboratory. As for earth dam and high embankment work where the soil will be placed under controlled conditions, triaxial shear tests are warranted.

b. Clays. For most foundation problems, unconfined compression tests on undisturbed samples provide the most practical determination of clay shear strength. The cohesion is taken as one-half the unconfined strength, and the angle of internal friction is ignored. Correlations have been made between unconfined strength and penetration resistance (Table 1-3). The in-place vane shear test (Section 3, Chapter 4) is valuable for determining the cohesion of very soft clays that may be excessively disturbed in sampling. Also, cohesion may be determined from a consolidated direct shear test, which is further useful when the anticipated gain in shear strength with consolidation is desired.

c. Silts and Mixed Soils. Procedures for evaluating the shear strength of these soils are not well formulated because of the variety of changes that can take place when draining under load. A practical, conservative approach is to determine the cohesion from unconfined tests. If the cohesion is substantial, the friction component may be ignored. If the unconfined strength is low, the soil can be treated as a granular soil, and the angle of friction can be estimated or determined from direct shear tests. The action under load and drainage in place must always be considered in making these determinations. Where the difficulty of the job and the uniformity of the soil warrant, triaxial tests may be performed under drainage conditions similar to those in the field. In cases of very soft silts, such as in marine deposits, the in-place vane shear test is especially helpful in evaluating the limited shear strength and its increase with depth.

Section 6. TESTS ON COMPACTED SOILS

1. MOISTURE-DENSITY RELATIONSHIPS. The Proctor test or a variation is employed in determining the moisture-density relationship. Proctor tests, however, are not applicable for cohesionless soils.

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. Specifically applicable to either a heavily compacted base course or a subgrade for airfield pavement, the modified Proctor test should not be used for mass earthwork, when such earthwork is compacted by ordinary methods because it yields an optimum moisture content that, almost without exception, is drier than desirable.

c. Relative Density of Cohesionless Soils. Proctor tests are inappropriate for free draining cohesionless soils and may give erratic compaction curves or densities substantially less than those provided by ordinary compaction in the field. No test for maximum and minimum densities is universally accepted. Use ASTM method D2049T (Table 3-3).

2. STRUCTURAL PROPERTIES. Structural properties of compacted-fill materials classified in the Unified System are listed in Table 9-1.

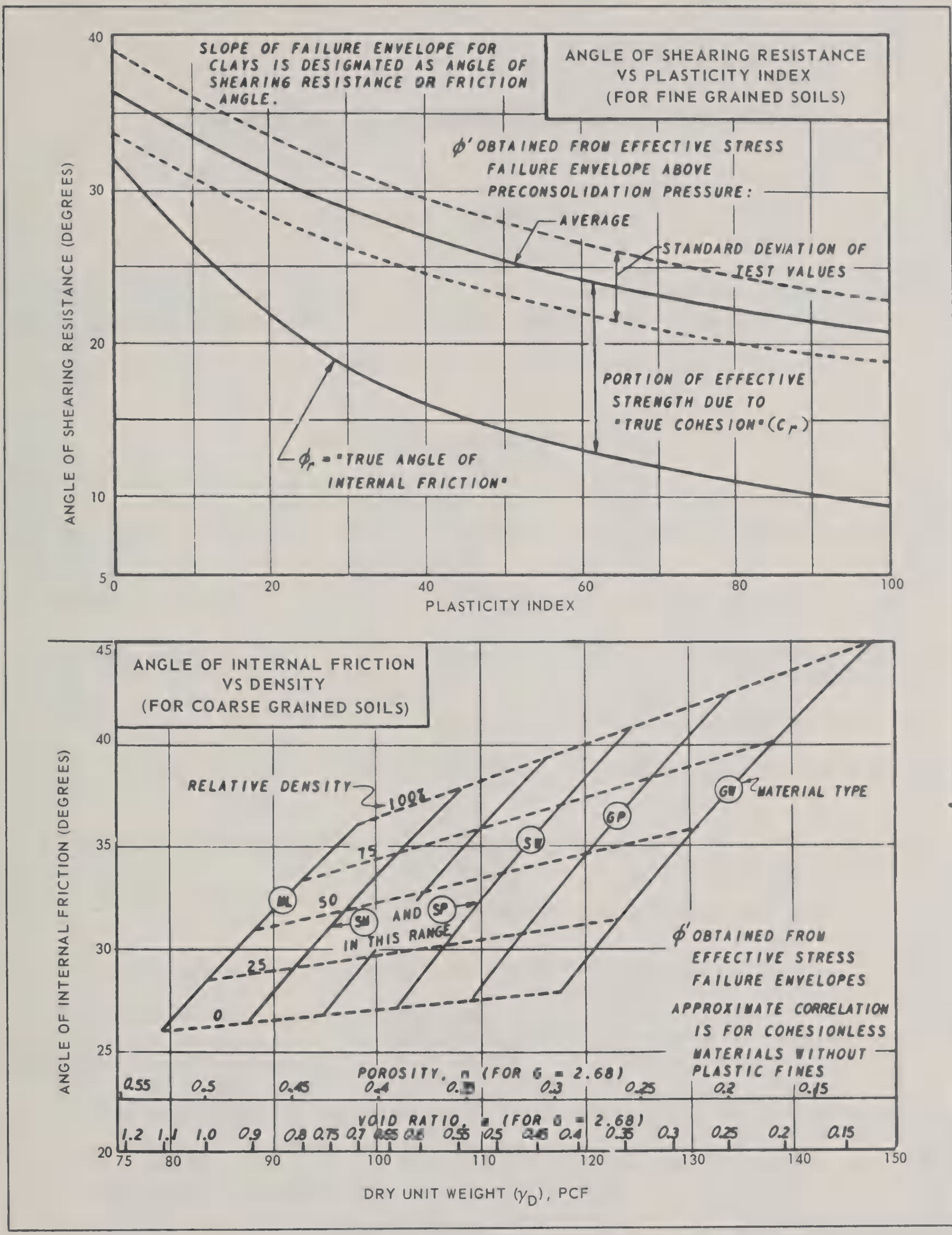


FIGURE 3-7
Correlations of Strength Characteristics

CHAPTER 4. FIELD TESTS AND MEASUREMENTS

Section 1. INTRODUCTION

1. **SCOPE.** This chapter describes procedures for field observations and measurement of soil properties in situ. Methods of performing plate bearing and pile tests are recommended. Certain techniques are based on methods given in the criteria sources cited. In other instances test procedures must be varied for requirements of the particular project.

Section 2. FIELD OBSERVATIONS

1. **UTILIZATION.** Observations include measurement of vertical or horizontal movement, ground water, or soil pressures for these purposes: To determine conditions existing at the start of construction for use in design; to control construction procedures or rate of construction; to predict the performance of the completed structure.

2. **VERTICAL MOVEMENT.** Observations of surface and subsurface settlement must be made.

a. Surface Settlement. Protect observation points against damage from construction activities or from frost heave. Provide reference bench marks on materials known to be incompressible, on rigidly supported structure, or on pile or pipe driven to refusal.

(1) Observation Points. Provide suitable settlement observation points within structures. Establish these points as soon as practicable during construction.

(2) Measuring Elevations. In most cases measurement of elevations to one-hundredth of a foot is sufficiently accurate.

b. Subsurface Settlement. Measurements are made of the settlement at the base of fill or at points within compressible strata. Use these observations in construction of earth dams, for fills over sand drains, or to evaluate heave of foundation excavation.

(1) Settlement Plate. Place settlement plate with riser pipe attached at base of fill. See Table 4-1.

(2) Probes. Drive probes into subsoil to measure foundation movement or push probe below bottom of test boring. See Table 4-1.

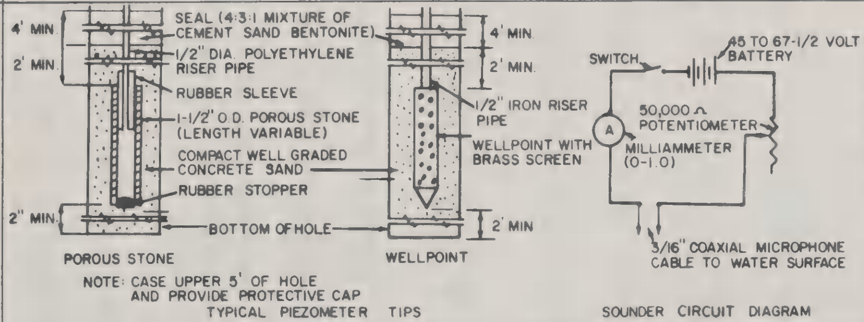
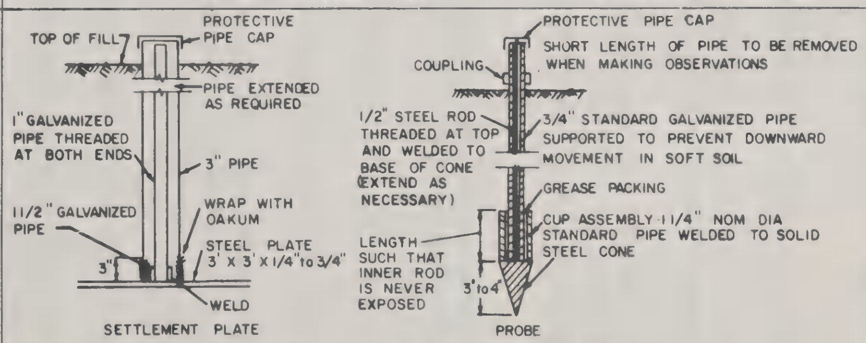
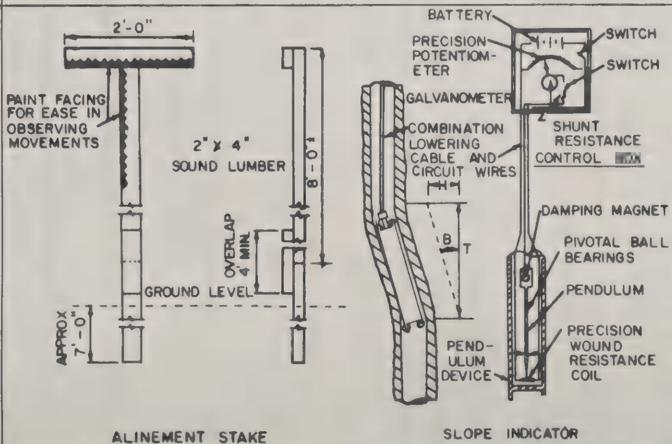
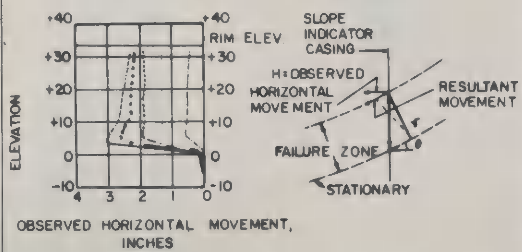
(3) Crossbars. For important earth dams, place a string of telescoping pipes with crossbars attached at vertical intervals in the fill as it is constructed. Measure movement of crossbars by special torpedo device lowered into the pipe. Use procedure in USBR, *Earth Manual*, Method E-29.

3. **HORIZONTAL MOVEMENT.** These measurements are made in connection with stability problems.

a. Soft Foundations. To control rate of filling on soft foundations, place T-stakes near embankment toe and observe horizontal movement. See Table 4-1.

b. Slide Areas. Install a slope indicator in active or potential slide areas to determine failure plane location and speed of shear. See Table 4-1. Compare shear strain observed in failure zone with shear strain in laboratory triaxial test to estimate strength mobilized in situ. See Figure 3-6.

TABLE 4-1
Equipment for Field Observations

Factor observed	Observation equipment	Operation and data obtained
Ground water level and pore water pressure		PIEZOMETERS Piezometer point consisting of porous tube or wellpoint is lowered in a completed borehole with a standpipe attached reaching to the ground surface. Sand filter is placed around the intake point, and intake point is isolated from contact with upper water levels by an impervious seal. Elevation of water in standpipe is measured by a sounder consisting of a shielded microphone cable whose tip is bared with the center conductor separated from the metal shield. Battery is connected between shield and center conductor. Milliammeter reacts abruptly when sounder tip reaches water level in standpipe. Elevation is measured to the nearest 0.01 ft. Elevation of tide or nearby open water should be noted at time of reading. Test the completed piezometer by raising or lowering standpipe water level and allowing it to fall to equilibrium. If it returns rapidly to the equilibrium position, the piezometer is sensitive and is responding to pore water pressures.
Vertical movement		SETTLEMENT PLATE Settlement plates are placed at base of fill or at intermediate heights in the fill during construction where movement is to be observed. Vertical movement of riser pipe usually is measured to nearest 0.01 ft. Time intervals between measurements depends on rate of fill construction. VERTICAL PROBE To determine settlement of points within foundation strata, drive probe to desired elevation using a drop hammer on the 3/4-in. pipe. Withdraw 3/4-in. pipe to prevent its bearing on conical point. Vertical movement of 1/2-in. steel rod usually is measured to nearest 0.01 ft.
Horizontal movement	 INSTALLATION Plastic casing, grooved longitudinally on two orthogonal diameters, is lowered in completed borehole. Space outside of casing is backfilled. Profile of slopes at vertical intervals is determined by lowering pendulum device in boring with guide rollers riding in grooves. Horizontal movement determined by integrating slopes from a point assumed fixed at bottom of casing.	ALINEMENT STAKE Used to observe horizontal movement of the ground surface where applied load may cause displacement of foundation materials. For use with fill construction, position stakes at 100 ft intervals on one, two, or three lines parallel to toe of fill slope. Observe movements from fixed reference line with both vertical and horizontal control at intervals during construction of fill. Measurements usually made to nearest 0.01 ft. SLOPE INDICATOR Used to determine location of failure zone and rate of shear strain within failure zone in active or potential slide areas.  TYPICAL SLOPE INDICATOR OBSERVATIONS

c. Earth Dams. For important earth dams, connect vertical plates in the fill with the telescoping pipe system to observe horizontal strains. Use procedure in USBR, *Earth Manual,* Method E-30.

4. TOTAL PRESSURES. These measurements are made to determine total pressures.

a. Pressure Cells. Combined earth and water pressures are measured with pancake cells having strain gages attached to an interior flexible diaphragm. Their installation and operation involves technical problems requiring specialized assistance. See Taylor, *Pressure Distribution Studies on Soils*, for description of pressure cells and interpretation of observations.

b. Indirect Measurements. Pressures on flexible wall are determined indirectly by measuring strain in sheeting or bracing system. Devices used include electrical strain gages or accurate mechanical gages. Their use involves technical problems requiring specialized assistance.

5. GROUND WATER PRESSURES. Pressures may be encountered in the following conditions: (a) hydrostatic state with insignificant seepage, (b) in a flow field of steep gradients, possibly with perched or depressed water levels, or (c) as hydrostatic excess pore pressures built up in materials of low permeability by volume change under load.

a. Hydrostatic Conditions. Few refinements are required in measuring techniques for these conditions. Observe water levels in test borings, allowing sufficient time for water to reach an equilibrium position. In material of low permeability, determine equilibrium level by raising or lowering water in borehole until no tendency for movement to equilibrium is observed.

b. Seepage Conditions. Where piezometric head varies greatly on a vertical line because of seepage, ordinary open borings or observation wells uncased for their entire length may be unsuitable for measurement. Water pressures in the stratum of greatest permeability, which delivers the largest flow to the hole, will dominate the observations. Utilize porous tube or wellpoint piezometers (Table 4-1). For materials and installation of porous tube piezometers, see USBR, *Earth Manual*, Method E-28. Seal the intake point of piezometer from contact with upper water levels to isolate observation to a specific point in the flow field.

c. Hydrostatic Excess Pressures. For rapid response to pore pressure changes, minimize the flow of water into the piezometer required by providing a large intake point and small diameter standpipe. Porous tube piezometers are sensitive enough to follow pressure changes in many clays with insignificant time lag.

(1) High Pore Pressure. To observe high pore pressure, attach Bourdon gage near top of standpipe with vent for bleeding air or gas above gage.

(2) Measurements. To avoid inaccurate measurements caused by gas pressures in organic soil, use piezometer seamless and jointless plastic standpipe with ID no less than 3/8 inch. Do not use metals in piezometer point. Do not permit water level in borehole or in piezometer to fall below pore pressures in soil surrounding the piezometer point.

(3) Special Installations. For special piezometer installations within high earth dams, see USBR, *Earth Manual*, Method E-27.

Section 3. MEASUREMENT OF SOIL PROPERTIES IN SITU

1. VANE SHEAR TEST. Equipment setup for the vane shear test is illustrated in Figure 4-1.

a. Applications. In situ vane shear tests are especially useful in very soft soil deposits where much of the strength may be lost by disturbance during sampling.

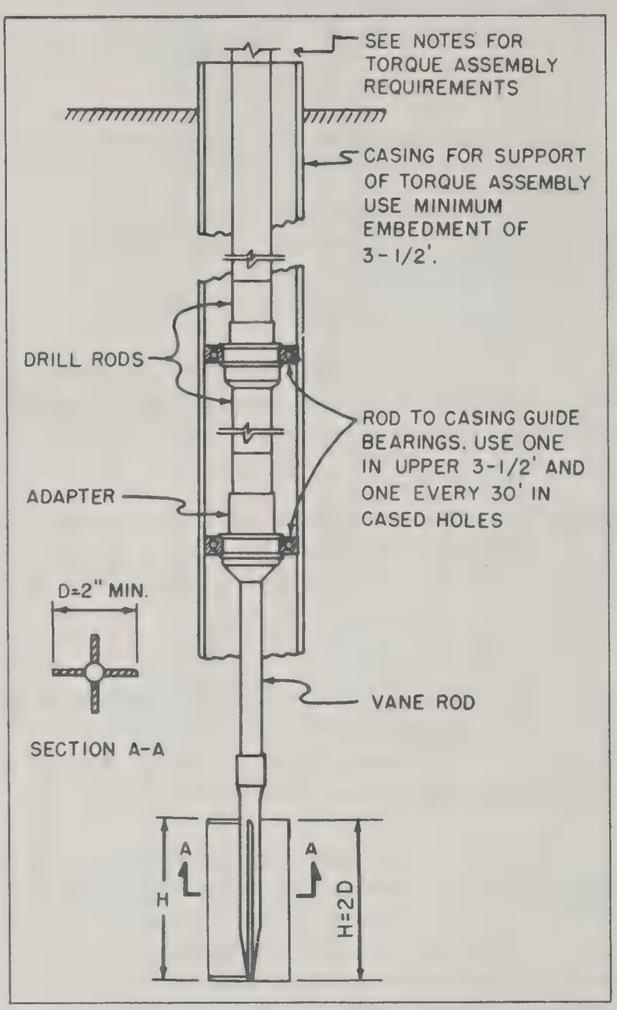


FIGURE 4-1
Equipment for Vane Shear Test

(1) *Stiff Clays*. Use is inappropriate in stiff clays where strength is controlled by fractures or slickensides.

(2) *Soft Soils*. Erratic results are obtained in soft soils containing shells, gravel, or wood fragments.

b. **Torque Assembly**. Requirements for the torque assembly include a gear reduction device capable of producing constant angular rotation of 1° to 6° per minute, a calibrated proving ring with a dial gage for force measurement, a means of measuring angular rotation in degrees, and thrust bearings to support vane at ground surface. Bearings shall be of the type arranged to minimize friction while preventing lateral movement of the vane. The gear reduction and proving ring device shall be calibrated to measure applied torque with an error of less than 5 percent.

c. **Procedure**. The following steps describe the procedure for the vane shear test.

(1) *Advancing Borehole*. Using rotary or wash boring methods, advance borehole with or without casing to a point no closer than 18 inches above intended test elevation, maintaining as close to vertical alinement as practical. Use cleanout procedure specified for undisturbed sampling in Paragraph 6, Section 9, Chapter 2, and maintain hole full of drilling fluid prior to and during test.

(2) *Advancing Vane*. Slowly lower vane to bottom of hole and push or jack the additional 18 inches, recording force necessary to advance vane.

(3) Recording Maximum Torque. Immediately after vane is in position with torque assembly in place, rotate vane at constant speed of 1° per minute and record maximum torque required.

(4) Obtaining Remolded Strength. To obtain remolded strength, rotate vane at rate of one revolution in 10 seconds for minimum of 12 revolutions without taking readings. Allow time delay of 5 minutes and perform Step 3 again.

(5) Obtaining Friction Resistance. Obtain friction resistance by proceeding as in Steps 2 and 3, using dummy rod without vane attached. Make one determination in each boring for vane shear tests.

(6) Identification Sample. After withdrawing vane from hole, remove and preserve a sample of the soil adhering to the vane for identification tests.

(7) Interpretation of Test Data. Determine shear strength by

$$S = \frac{6\,(T_{max} - \text{friction resistance})}{\pi D^2\,(3H + D)}$$

where T_{max} = maximum torque in inch-pound as obtained in Step 3 of Procedure above, H = vane height in inches, D = vane diameter in inches, and S = shear strength in psi. Friction resistance is obtained in Step 5 of Procudure above.

2. PENETRATION RESISTANCE. Perform standard penetration resistance test in ordinary exploration as a routine identification procedure (Table 4-2).

a. Coarse Grained Soils. Correlations between penetration resistance, relative density, and effective overburden pressure are presented in Figure 4-2. Use the indicated relative density in estimating structural properties. Increased length of drill rod and change in rod weight have relatively small effect on the blow count compared to change in overburden pressure. Drill rod variations are ignored in approximate correlation of Figure 4-2.

b. Fine Grained Soils. See Figure 4-2 for correlations of unconfined compressive strength with penetration resistance. In general, strength for a given blow count is greater for the more plastic materials. For clayey silts, CL clays, or varved clays and silts, use: q_u (tsf) = 0.15 × standard penetration resistance. For CH clays use: q_u (tsf) = 0.20 × standard penetration resistance.

c. Limitations. Except where confirmed by specific structural properties tests, these relationships are suitable for preliminary estimates only. Blow counts in sands are increased by the presence of large gravel. Penetration resistance in clays does not reflect fractures or slickensides, which may be important to strength characteristics.

3. PERMEABILITY. See Table 4-2 for field test procedures.

a. Pumping Test. This test requires installation of pumping well plus piezometers to observe drawdown water levels on radial lines from the well. Use pumping tests to predict requirements for large-scale dewatering of construction site or permanent pressure relief of large structures. Observation wells or piezometers should conform to requirements of Section 2. For analysis of pumping well drawdown, see Zanger, *Theory and Problems of Water Percolation* (Bibliography).

b. Variable Head Permeability Test. These tests estimate permeability in situ by means of piezometers observation wells. Standpipe water level is raised or lowered from equilibrium and allowed to recover while readings are made of elevation versus elapsed time. See Figure 4-3 for analysis of observations. Compute permeability, using shape factors of Table 4-3. For tests in anisotropic materials, transform dimensions of intake point of piezometer or observation well as shown in Figure 4-3. Estimate horizontal permeability by trial, using piezometers with intake points of different shape. Assume various ratios of horizontal to vertical permeability determined from several piezometers is made equal.

TABLE 4-2
Measurements of Soil Properties in Situ

Property determined and name of test	Test application	Test equipment and procedure
Penetration resistance:		
Standard penetration test .	Routine procedure in test borings, performed for each dry sample obtained with standard split spoon.	Use equipment and procedure of ASTM D-1586 with the following revisions: 1) Provide sampler with 20-in.-long split barrel. When exceeding 30 blows per foot, 12-in. minimum length of drive is permissible. 2) Record blows for each 6 in. of penetration. 3) Maintain hole full with water or drilling fluid during driving.
Permeability:		
Pumping test	Applied for evaluation of problem of large scale drawdown or dewatering in materials with substantial variation in permeability.	Rate at which water is pumped from a central test well is measured while the drawdown or radial lines extending from the well is observed in a series of piezometers or observation holes. Provide 3 to 5 observation wells spaced at increasing intervals along two radial lines separated by 90° central angle.
Variable head test	Used to obtain individual permeability determinations in piezometers. Performed in porous tube or wellpoint piezometers that have been placed in boreholes with their intake point isolated by impervious seal.	Standpipe water level is raised or lowered from its equilibrium position and readings are taken of water levels at periodic intervals as it returns to equilibrium. Observations of differential head and time elapsed are analyzed as shown in Figure 4-3.
Borehole test...........	Used to determine permeability of individual strata penetrated by borings.	May be performed in a cased, open-end borehole or an uncased borehole with double packers. Rate at which water flows out of the borehole under a constant head is measured. Use equipment and procedures of USBR Method E-18.
Auger hole test	Performed in shallow, uncased auger hole. May be used in unsaturated material where the water table during testing is at a great depth below the base of hole.	Rate at which water flows out of the uncased hole is measured maintaining a constant water level in the hole. Use equipment and procedures of USBR Method E-19.
Dry unit weight and moisture content:		
Sand density cylinder....	Used in soil of any type but least accurate in clean, gravelly, or rocky materials.	Dry unit weight and moisture content of soil removed from hand excavated hole is determined, measuring volume of hole by sand displacement. Use equipment and procedures of USCE EM 1110-45-302, Appendix III.
Drive sample method	Used in cohesive, fine grained soils. Method has less chance for error and is more rapid than sand density cylinder procedure.	Dry unit weight and moisture content is determined on sample obtained from a thin-wall tube which is driven or pressed into the soil. Use equipment and procedure of USCE EM 1110-45-302, Appendix III.

c. **Borehole Permeability Test.** See Table 4-2. Use these tests in open-end or uncased boreholes.

d. **Auger Hole Test.** See Table 4-2. Use these tests in auger boring or uncased hole to determine permeability of materials near ground surface above existing water table. Square or rectangular test pits

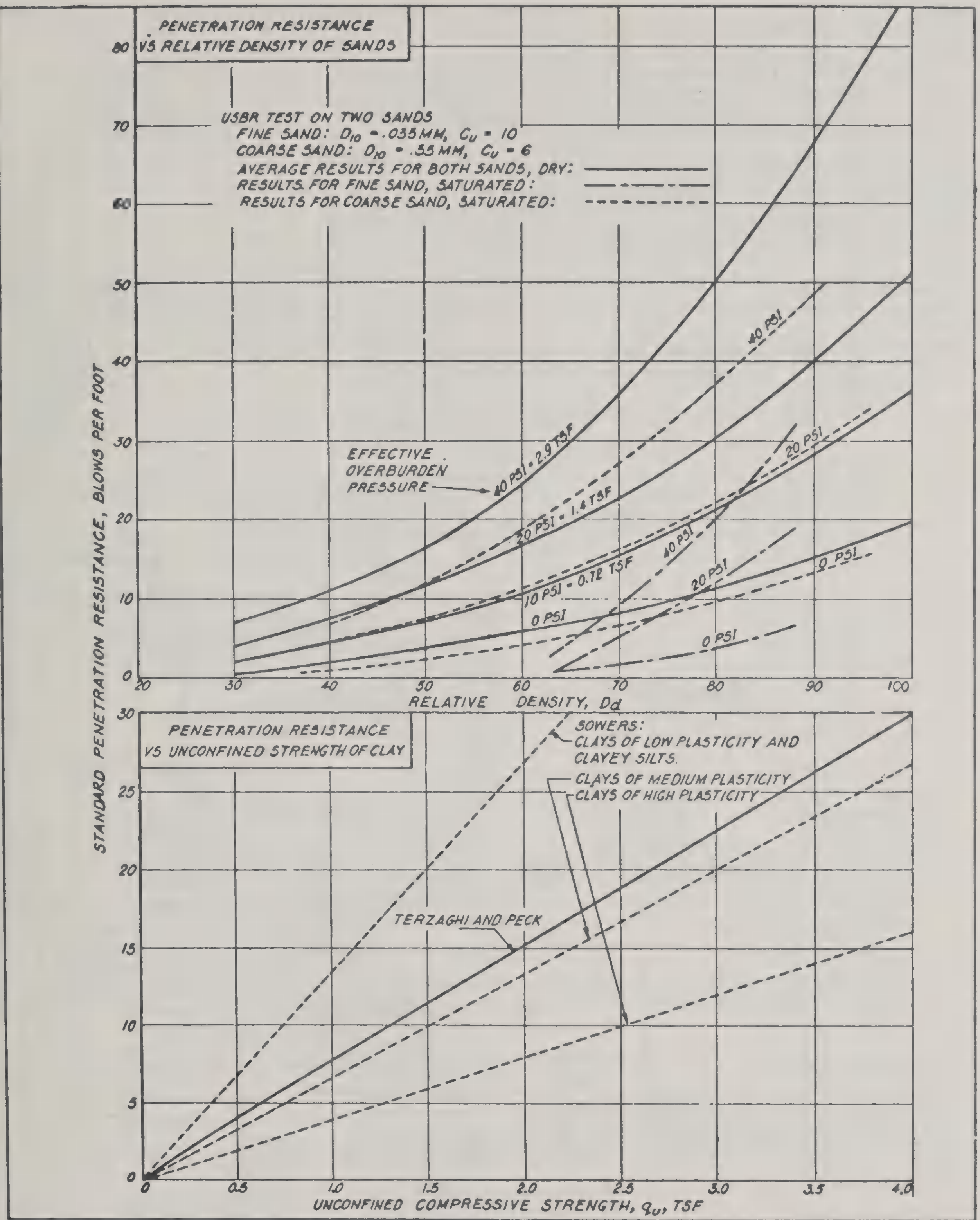
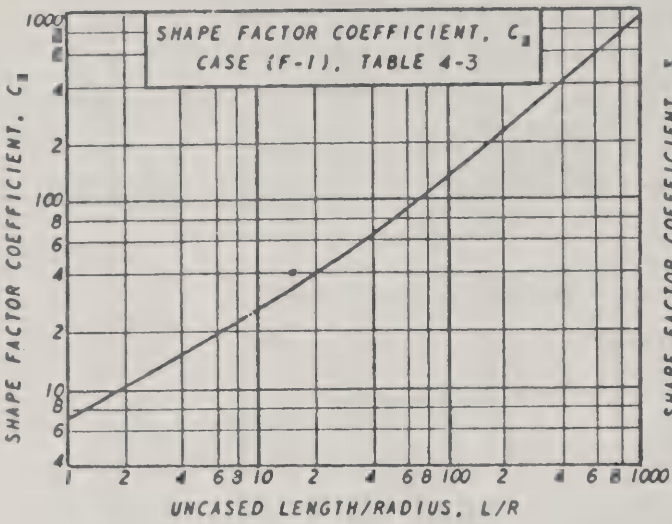
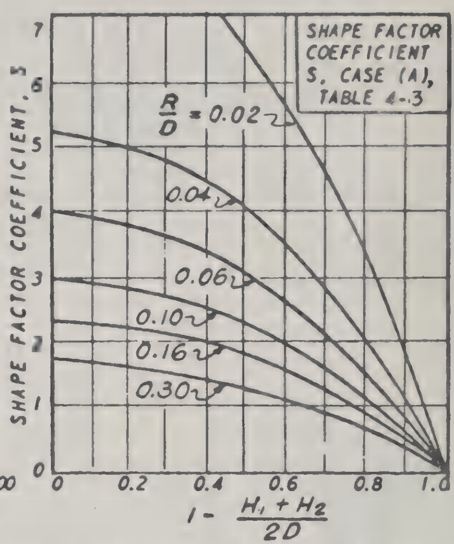
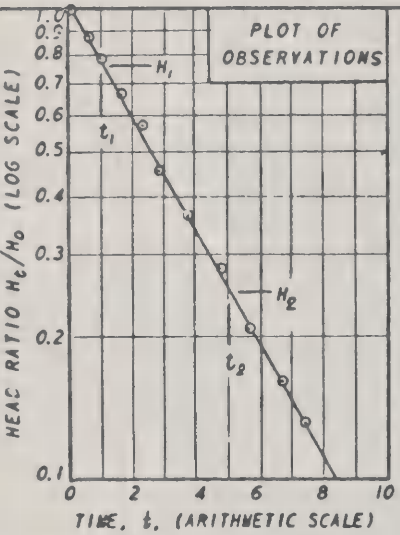


FIGURE 4-2
Correlations of Standard Penetration Resistance



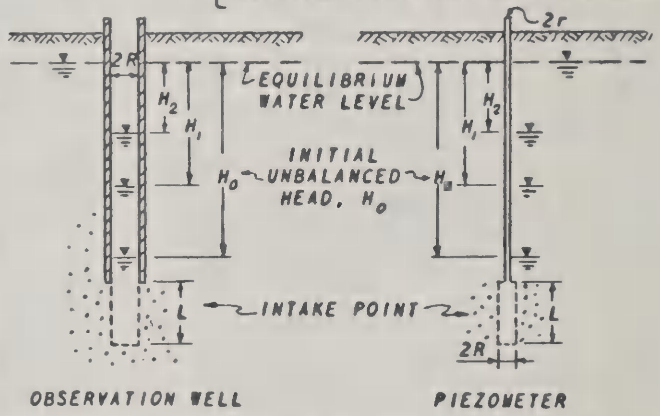




IN GENERAL:

$$K = \frac{A}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$

F = SHAPE FACTOR OF INTAKE POINT
A = STANDPIPE AREA
K = MEAN PERMEABILITY
Ln H_1/H_2 AND $(t_2 - t_1)$ ARE OBTAINED FROM PLOT OF OBSERVATIONS.



OBSERVATION WELL IN ISOTROPIC SOIL:

OBTAIN SHAPE FACTOR FROM TABLE 4-3.

FOR CASE (C):

$$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$$

$$K = \frac{R^2}{2L} \ln\left(\frac{L}{R}\right)\left[\frac{\ln H_1/H_2}{(t_2 - t_1)}\right]$$

PIEZOMETER IN ISOTROPIC SOIL:

RADIUS OF INTAKE POINT (R) DIFFERS FROM RADIUS OF STANDPIPE (r).

$$F = \frac{2\pi L}{\ln\left(\frac{L}{R}\right)}$$

$$A = \pi r^2$$

$$K = \frac{A}{F(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$$

$$K = \frac{r^2}{2L} \ln\left(\frac{L}{R}\right)\left[\frac{\ln H_1/H_2}{t_2 - t_1}\right]$$

TEST IN ANISOTROPIC SOIL:

ESTIMATE RATIO OF HORIZONTAL TO VERTICAL PERMEABILITY AND DIVIDE HORIZONTAL DIMENSIONS OF THE INTAKE POINT BY:

$m = \sqrt{K_H/K_V}$ TO COMPUTE MEAN PERMEABILITY $K = \sqrt{K_H \times K_V}$.

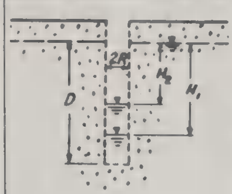
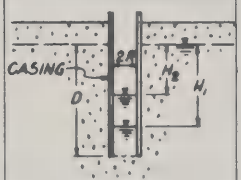
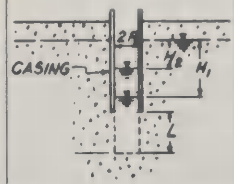
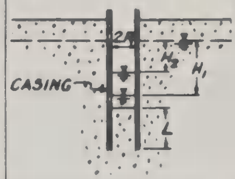
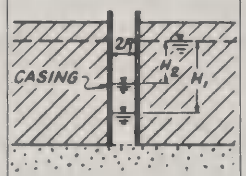
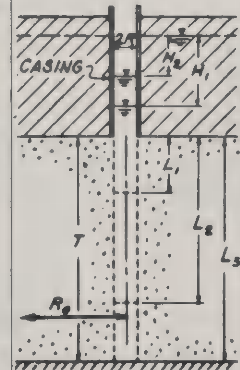
FOR CASE (C), TABLE 4-3:

$$F = \frac{2\pi L}{\ln\left(\frac{mL}{R}\right)}$$

$$K = \frac{r^2}{2L} \ln\left(\frac{mL}{R}\right) \frac{\ln H_1/H_2}{(t_2 - t_1)}$$

FIGURE 4-3
Analysis of Permeability by Variable Head Tests

TABLE 4-3
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 4-3.
	(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".		$F = \frac{11\pi R^2}{2\pi R + 11L}$	$K = \frac{2\pi R + 11L}{11(t_2 - t_1)} \ln \frac{H_1}{H_2}$	Principal use is for permeability in vertical direction in anisotropic soils.
OBSERVATION WELL OR PIEZOMETER IN AQUIFER WITH IMPERVIOUS UPPER LAYER	(E) Cased hole, opening flush with upper boundary of aquifer of infinite depth.		$F = 4R$	$K = \frac{\pi R}{4(t_2 - t_1)} \ln\left(\frac{H_1}{H_2}\right)$	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) $\frac{L_1}{T} \leq 0.20$ (2) $0.2 < \frac{L_2}{T} < 0.85$ (3) $\frac{L_3}{T} = 1.00$ Note: R_o 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 5 ft. For values of C_s, see Figure 4-3.
			(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 $\frac{L_2}{R} > 8$	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 $\frac{R_o}{R}$ = 200 for estimates unless obser ations wells are made to determine actual value of R_o.

may be utilized for the test. See Zanger, *Theory and Problems of Water Percolation* (Bibliography), for conversion of test pit dimensions to an equivalent circular hole.

4. IN-PLACE DENSITY. Tests are performed for embankment construction control or to estimate properties of foundation soils. Determine density in hand excavated holes at the ground surface or by recovery of undisturbed samples in boring operations.

a. Surface Density Tests. Use sand density cylinder procedure of Table 4-2. In cohesive fine grained soils, the drive sample test procedure of Table 4-2, using a thinwall tube, may be more convenient than the sand density test. For very rocky or clean gravelly soils, no in-place density procedure gives completely reliable results. The various water balloon test procedures are less accurate than sand density methods and should not be used unless comparisons for specific materials indicate satisfactory results.

b. Subsurface Density Determinations. Undisturbed samples of fine grained cohesive soils from test borings provide satisfactory density determinations. To ascertain relative density of coarse grained soils at depth, specify the highest quality undisturbed sampling with drilling mud.

c. Nuclear Density Test Equipment. Commercial equipment is available for surface or depth measurement of density or moisture by neutron and gamma ray emission. Procedures are rapid, convient, and particularly useful for determinations in clean, coarse grained material. The initial cost of equipment is high. Results of individual tests may vary significantly from those obtained by conventional methods, but the averages of a large number of tests made by either method should be in agreement.

Section 4. PERFORMANCE TESTS

1. LIMITATIONS. Performance tests assist in determining soil properties in situ. Loading a single isolated pile or bearing plate applies stress to the foundation to a lesser extent than does the completed structure. Evaluate overall settlement and stability of the entire foundation, apart from load tests, by methods of Chapters 6 and 7.

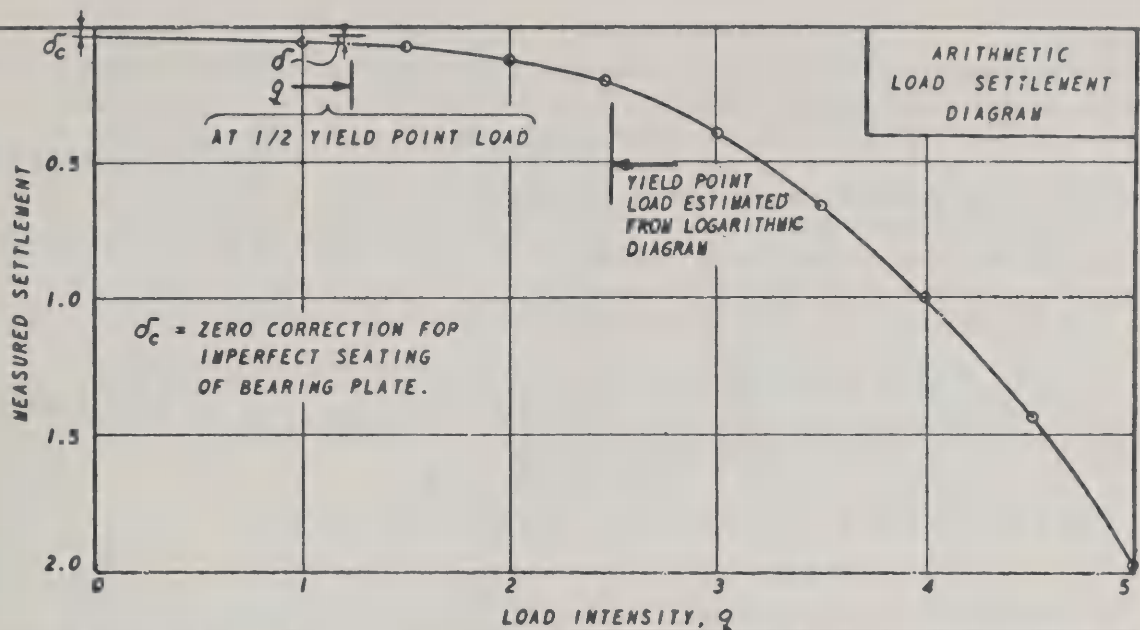
2. PLATE BEARING TEST.

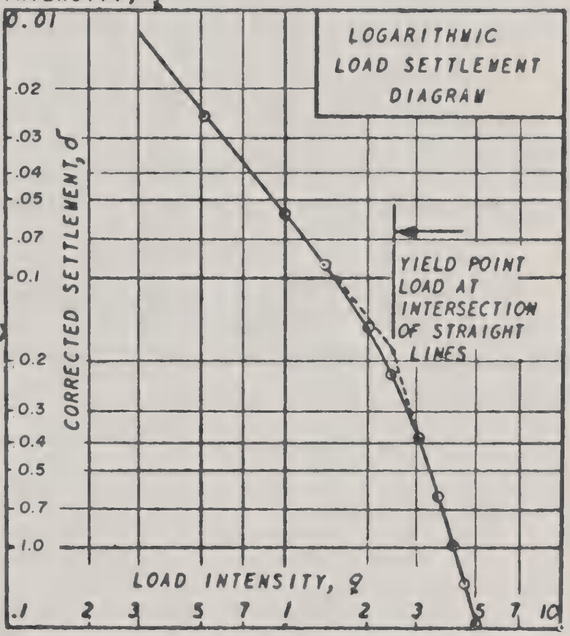
a. Procedure. For ordinary tests for foundation studies, use procedure of ASTM D 1194, except that dial gages reading to 0.001 in. should be substituted. Tests are utilized to estimate immediate settlements of spread foundations. Results obtained have no relation to deep seated settlement from volume change under load of entire foundation.

b. Analysis of Test Results. Determine yield point pressure from logarithmic plot of load versus settlement. Convert modulus of subgrade reaction determined from test k_v to the property k_{v1} for use in computing immediate settlement. In general, tests should be conducted with ground water and saturation conditions simulating those anticipated in the prototype. See Figure 4-4.

3. PILE DRIVING TEST. Perform driving resistance tests in any detailed study of pile foundations. Analyze exploration and laboratory test data to select appropriate pile type and bearing stratum before undertaking the driving test. Complete a driving test on all piles later subject to load tests.

a. Interpretation. See Chapter 13.





DEFINITIONS

K_{v1} = MODULUS OF SUBGRADE REACTION FOR 1-FT-SQUARE BEARING PLATE AT GROUND SURFACE.

K_v = MODULUS OF SUBGRADE REACTION FOR SQUARE BEARING PLATE OF ANY WIDTH B AT GROUND SURFACE.

q = APPLIED LOAD INTENSITY

δ = CORRECTED SETTLEMENT = MEASURED SETTLEMENT $-\delta_c$

B = WIDTH OF SQUARE BEARING PLATE (FT)

R = RADIUS OF CIRCULAR BEARING PLATE (FT)

E_s = MODULUS OF ELASTICITY OF SOIL

$C_1 C_2$ = LOAD TEST PARAMETERS

TO DETERMINE K_v:

1. ESTIMATE δ_c BY BACKWARD PROJECTION OF ARITHMETIC LOAD-SETTLEMENT CURVE TO ZERO LOAD.
2. PLOT LOGARITHMIC LOAD-SETTLEMENT CURVE AND DETERMINE YIELD POINT LOAD.
3. DETERMINE δ AND q AT 1/2 YIELD POINT LOAD.
4. $K_v = q/\delta$

TO DETERMINE K_{v1}:

FOR FIRM COHESIVE SOILS:

$K_{v1} = BK_v$ (SQUARE PLATE)

$K_{v1} = 2RK_v$ (CIRCULAR PLATE)

$E_s = 0.95\,K_{v1}(1-\mu^2)$

FOR COHESIONLESS COARSE GRAINED SOILS:

$K_{v1} = \frac{4B^2}{(B+1)^2}K_v$ (SQUARE PLATE)

$K_{v1} = \frac{16R^2}{(2R+1)^2}K_v$ (CIRCULAR PLATE)

FOR SOILS COMBINING FRICTION AND COHESION:

$K_v = \frac{q}{\delta} = C_1 + \frac{C_2}{B}$

PERFORM 2 TESTS ON PLATES OF DIFFERENT WIDTHS. SOLVE FOR PARAMETERS C_1 AND C_2.

$K_{v1} = C_1 + C_2$

NOTE: ABOVE RELATIONSHIPS APPLY AT SAME CONTACT PRESSURE.

FIGURE 4-4
Analysis of Plate Bearing Tests

b. Correlation. Correlate driving resistance with test boring data to predict performance of contract piles in various subsoil profiles of the site.

4. PILE LOADING TESTS. Use ASTM Method D1143. Do not use the D1143 (Figure 3) loading method of jacking against holddown piles where dead load reaction is feasible.

a. Allowable Load. Subject test piles to twice the anticipated design load. Use procedures of Figure 4-5 to determine design load. For instances where procedures of Figure 4-4 are not applicable, use for design 50 percent of the load that causes 1/4-inch permanent settlement.

b. Tip Load. Where negative friction may act on the pile, drive the test pile in a hole cased and cleaned to the bearing stratum, or measure movement of the tip directly by extension rods attached to the pile tip. Analyze by Figure 4-5, upper portion.

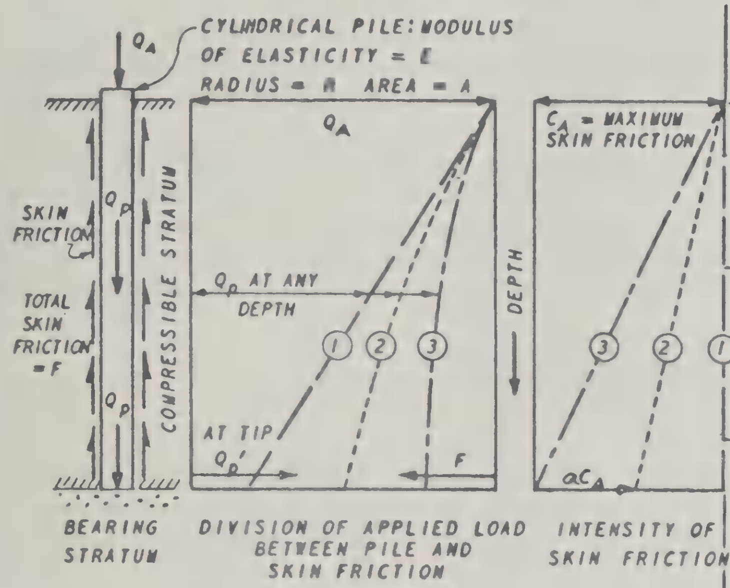
5. PILE PULLING TESTS. Perform pulling tests on piles previously subjected to driving resistance tests.

a. Procedure. As far as practicable, arrange the reaction to avoid increasing pullout resistance of test pile. Where this cannot be avoided entirely, make a conservative selection of allowable load.

b. Allowable Load. Criteria vary depending on tolerable movement of the structure. Failure load may be taken as that value at which upward movement suddenly increases disproportionately to load applied. Failure load may be taken as that value producing a net movement of butt equal to 1/4 in. Apply safety factor between 1-1/2 and 2 to failure load to obtain allowable working load.

6. TESTS OF BATTER PILE FRAMES. Under special circumstances it may be necessary to determine lateral load capacity of a frame of batter piles or the pullout capacity of a single pile of the frame, or both. For this purpose utilize ASTM Suggested Test Method, *Procedures for Testing Soils* (ASTM).

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 aC_A AT TIP:

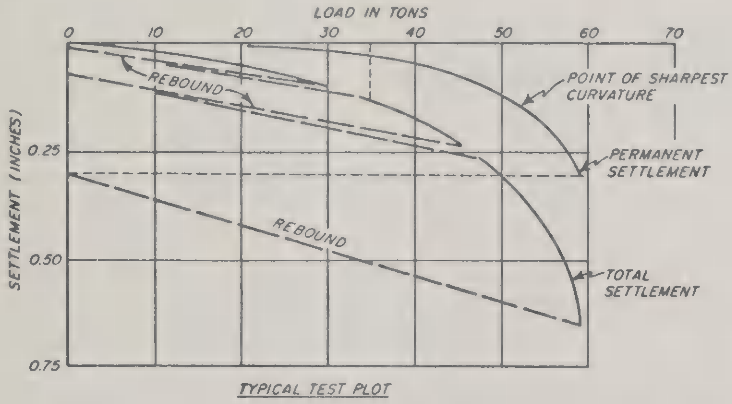
$$\delta_E = \left(Q_A - 2\pi R C_A L\left(\frac{a+2}{3(a+1)}\right)\right)\frac{L}{AE}$$

$$Q_p' = \frac{3AE\delta_E}{L}\left(\frac{a+1}{a+2}\right) - Q_A\left(\frac{2a+1}{a+2}\right)$$

CASE (3). SKIN FRICTION DECREASING TO ZERO AT TIP:

$$\delta_E = \left(Q_A - \frac{4\pi R C_A L}{3}\right)\frac{L}{AE}$$

$$Q_p' = \frac{3AE\delta_E}{2L} - \frac{Q_A}{2}$$



TYPICAL TEST PLOT

SELECT THE PERMISSIBLE DESIGN LOAD UNDER THESE TWO BASIC REQUIREMENTS:

(A) DO NOT APPROACH TOO CLOSELY TO THE YIELD POINT OF THE SOIL

(B) RESTRICT GROSS SETTLEMENT WITHIN LIMITS PERMISSIBLE FOR STRUCTURE

(1) MEET REQUIREMENT (A.) BY USING NOT MORE THAN 2/3 THE LOAD CORRESPONDING TO THE POINT OF SHARPEST CURVATURE ON THE PERMANENT SETTLEMENT CURVE.
EXAMPLE: 53 TONS × 2/3 = 35 TONS

(2) CHECK REQUIREMENT (B.) BY OBSERVING THE CORRESPONDING TOTAL SETTLEMENT.
EXAMPLE: TOTAL SETTLEMENT UNDER 35 TONS = 0.14"

(3) USE 35 TONS OR A LESSER LOAD IF THE SETTLEMENT REQUIREMENTS OF THE STRUCTURE SO DICTATE.

FIGURE 4-5
Analysis of Pile Loading Tests

CHAPTER 5. DISTRIBUTION OF STRESSES AND PRESSURES

Section 1. INTRODUCTION

1. **SCOPE.** This chapter covers analysis of stress conditions at a point, stress distribution in elastic foundations, and principal stresses in active or passive states.

2. **STATE OF STRESS.** Stresses in earth masses are analyzed by two general procedures. One assum elastic conditions, the other assumes full mobilization of shear strength. If the safety factor against sh failure exceeds about 3, stresses are roughly equal to values computed from elastic theory. Elastic solu tions apply to settlement problems for which shear failure is unlikely. Plastic equilibrium applies in pro lems of foundation or slope stability and wall pressures where shear strength may be completely mobiliz

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. See Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, for theoretical relationships.

a. **Plastic Equilibrium.** The use of Mohr's circle for plastic equilibrium is illustrated by analysis of triaxial shear test results in Figure 3-6.

b. **Elastic Solution.** See Figure 5-1 for application of Mohr's circle to the elastic solution for stresses beneath an infinitely long embankment. The pattern of normal and shear stresses on vertical and horizontal planes is derived by elastic analysis. Mohr's circle for three points A, B, and C in the foundation are plotted using these vertical and horizontal stresses. Magnitude and orientation of principal stresses at the three points may then be determined from Mohr's circle.

2. **EFFECTIVE AND NEUTRAL STRESSES.** Total normal stress at any orientation in the soil mass is divided into two elements; (a) effective stress, carried by grain skeleton of the soil, and (b) neutral stress or pore pressure carried by fluid in void spaces.

a. **Total Pressure.** Whether derived from body forces or superposed load, total pressure equals the sum of effective plus neutral stress.

b. **Effective Stress.** This stress is measured as total force in the grain skeleton divided by total cross-section area over which the force acts.

c. **Body Forces.** Division of body forces into effective and neutral stresses depends on position of ground water table or the flow field set up by seepage.

d. **Superposed Load.** Division of superposed load between neutral and effective stresses is a function of stress-strain properties of the soil and relief of pore pressures afforded by pore water drainage.

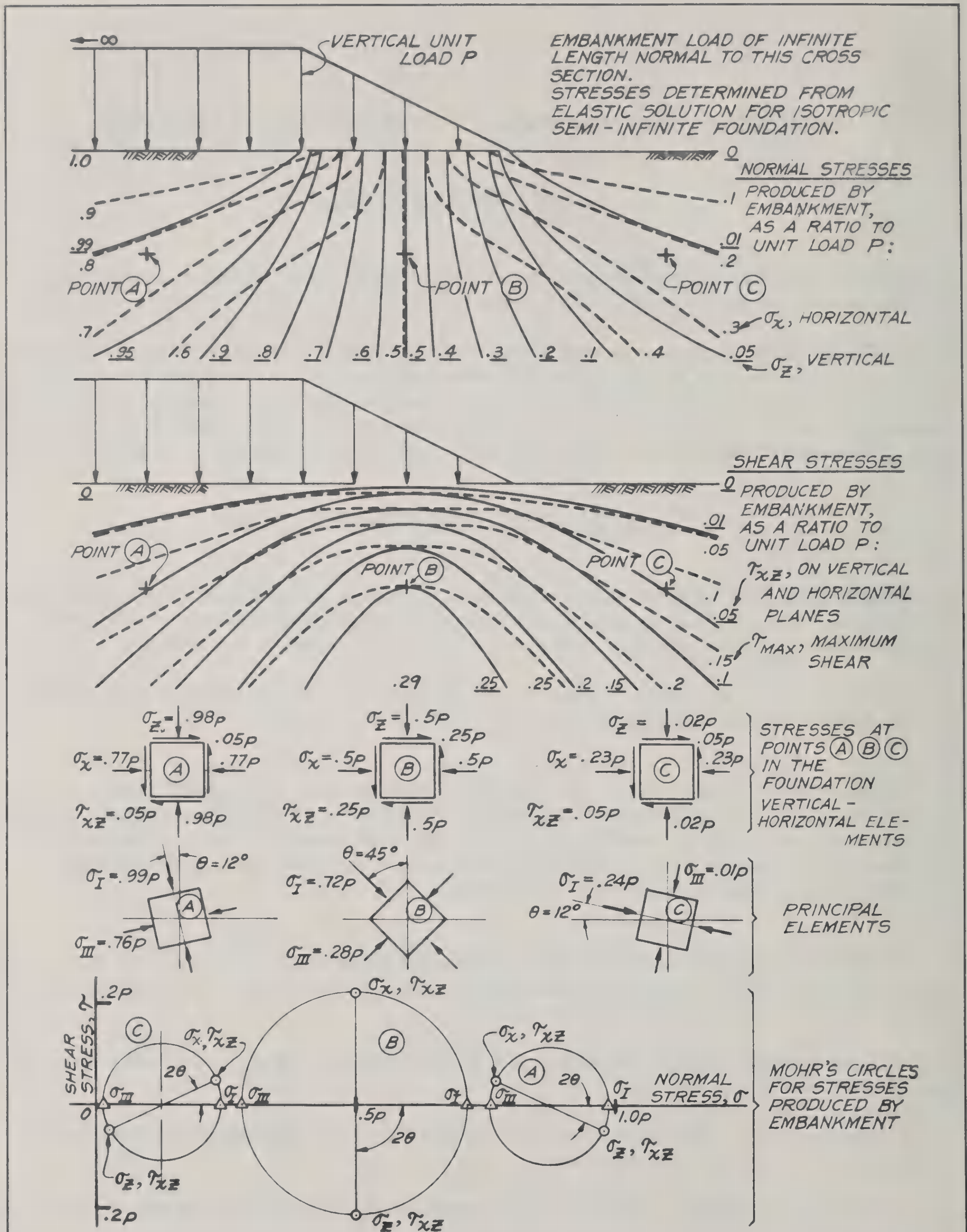


FIGURE 5-1
Examples of Stress Conditions at a Point

Section 3. STRESS DISTRIBUTION IN ELASTIC FOUNDATIONS

1. SEMI-INFINITE, ISOTROPIC FOUNDATIONS. Formulas for stresses produced by various loads on semi-infinite, elastic isotropic foundations are listed in Figure 5-2.

a. Assumed Conditions. These solutions assume elasticity, continuity, static equilibrium, and completely flexible loads so that pressures on the foundation surface equal load intensity. The formulas are designated as equations of the "Boussinesq" type. For loads of infinite length, the foundation undergoes plane strain; that is, deformation occurs only in planes perpendicular to the long axis of the load. In this case stresses depend only on shape of load and position in the foundation of points being investigated and are not affected by elastic properties.

b. Vertical Stresses Beneath Regular Loads. Influence values for vertical stresses under simple geometric loads are given in Figures 5-3 through 5-6. For examples of their use, see Figure 5-7. Computation procedures are as follows:

(1) Divide a rectangular mat foundation into four rectangles with their common corner above the point investigated. Obtain influence values I for the individual rectangles from Figure 5-3, and sum the values to obtain total I.

(2) Use Figure 5-4 to compute stresses produced by a group of individual footings that may be approximated by loaded circles.

(3) Figure 5-5 applies to stresses beneath an embankment of infinite length. Analyze fills of more complicated cross section by adding or subtracting simple forms 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.

(4) Obtain stresses beneath sloping fill of finite dimension from Figure 5-6.

c. Vertical Stresses Beneath Irregular Loads. Use Figure 5-8 for complicated loads where the influence diagrams do not suffice. Proceed as follows:

(1) Draw a circle of convenient scale and the concentric circles shown within it.

(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.

(4) See the bottom chart for influence values I for stresses at various depths produced by the loads within each annular space. The product $I \times A$ 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.

d. Horizontal Stresses. Elastic analysis of horizontal stresses is utilized to determine pressures on unyielding walls from surcharge loads, and pressures on rigid buried structures. See basic formulas for simple loads in Figure 5-2. Applications are discussed in Chapters 10 and 14.

e. Shear Stresses. Elastic solutions generally are not applicable where shear stresses are critical, as in stability problems. To estimate if the complete stability analysis described in Chapter 7 is required, determine maximum shear stresses produced by the applied load from the elastic formulas and compare this stress with available shear strength. For embankment loads in Figure 5-2 maximum shear stress in the foundation is exactly or approximately equal to p/π. If load intensity equals π times shear strength s, plastic conditions prevail at a single point in the foundation. As the load increases beyond this value, a larger and larger portion of the foundation passes into plastic equilibrium. In this case failure is possible and overall stability must be evaluated.

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University of Illinois
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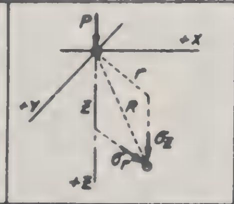
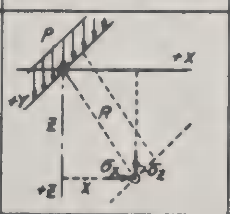
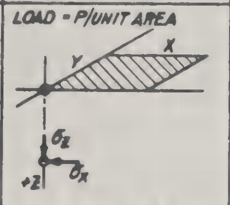
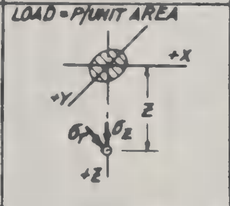
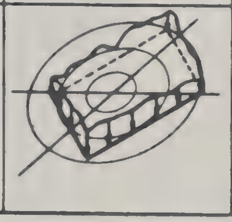
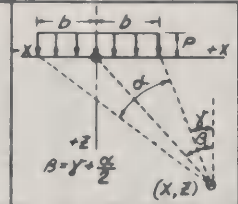
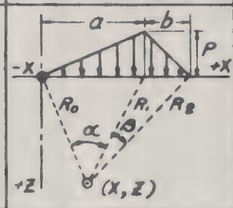
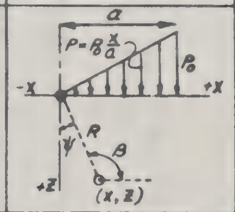
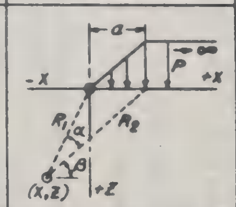
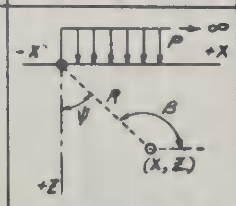
Urbana, Illinois

Loading condition	Stress Diagram	Stress component	Equation
Point load		Vertical	$\sigma_z = \frac{3P}{2\pi}\cdot\frac{z^3}{R^5}$
		Horizontal	$\sigma_r = \frac{P}{2\pi}\left[3\frac{r^2z}{R^5} - (1-2\mu)\left(\frac{R-z}{Rr^2}\right)\right]$
		Shear	$\tau_{rz} = \frac{3P}{2\pi}\cdot\frac{rz^2}{R^5}$
Uniform line load of infinite length		Vertical	$\sigma_z = \frac{2p}{\pi}\;\frac{z^3}{R^4}$
		Horizontal	$\sigma_x = \frac{2p}{\pi}\cdot\frac{x^2z}{R^4}$
		Shear	$\tau_{xz} = \frac{2p}{\pi}\cdot\frac{xz^2}{R^4}$
Uniformly loaded rectangular area (Fig. 5–3)		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 (Fig. 5-4)		Vertical	$\sigma_z = p\left[1-\left(\frac{1}{1+(r/z)^2}\right)^{3/2}\right]$
		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 σ_z, σ_r, τ_{rz} BENEATH CENTER OF CIRCLE)
Irregular load		Vertical	COMPUTED FROM INFLUENCE CHART OF FIG. 5 - 8.

Embankment loads of infinite length

Loading condition	Stress diagram	Stress component	Equation
Uniform strip load		Vertical	$\sigma_z = \frac{p}{\pi}[\alpha+\sin\alpha\cdot\cos2\beta]$
		Horizontal	$\sigma_x = \frac{p}{\pi}[\alpha-\sin\alpha\cdot\cos2\beta]$
		Shear	$\tau_{xz} = \frac{p}{\pi}\sin\alpha\cdot\sin2\beta$
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	$\sigma_z = \frac{p_0}{\pi a}[x\beta+z]$
		Horizontal	$\sigma_x = \frac{p_0}{\pi a}[x\beta-z-2z\log_e R]$
		Shear	$\tau_{xz} = \frac{p_0}{\pi a}z\beta$
Terrace load		Vertical	$\sigma_z = \frac{p}{\pi a}[a\beta+x\alpha]$
		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 5-2
Formulas for Stresses in Semi-Infinite Elastic Foundation

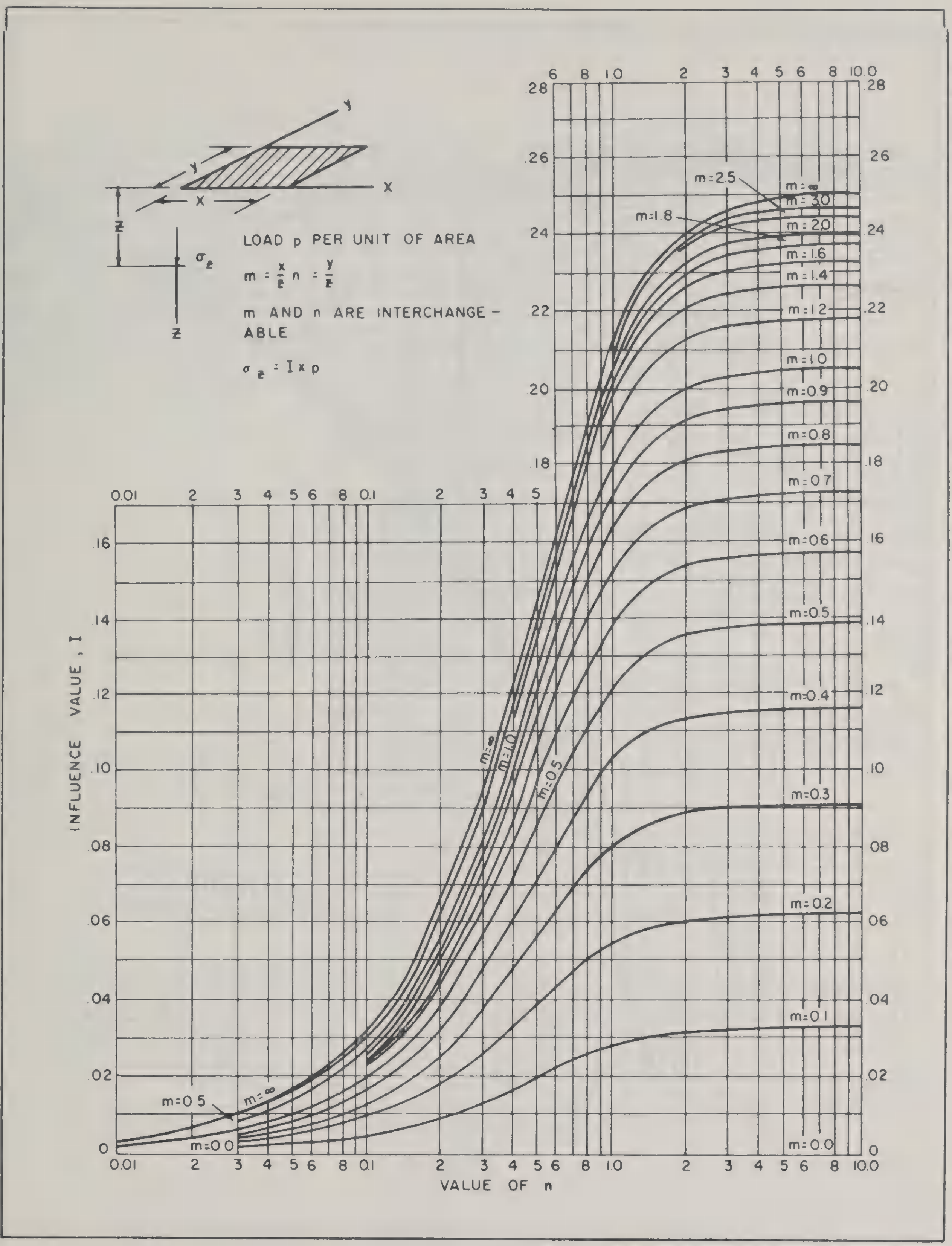


FIGURE 5-3
Influence Value for Vertical Stress Under Corner of Uniformly Loaded Rectangular Area

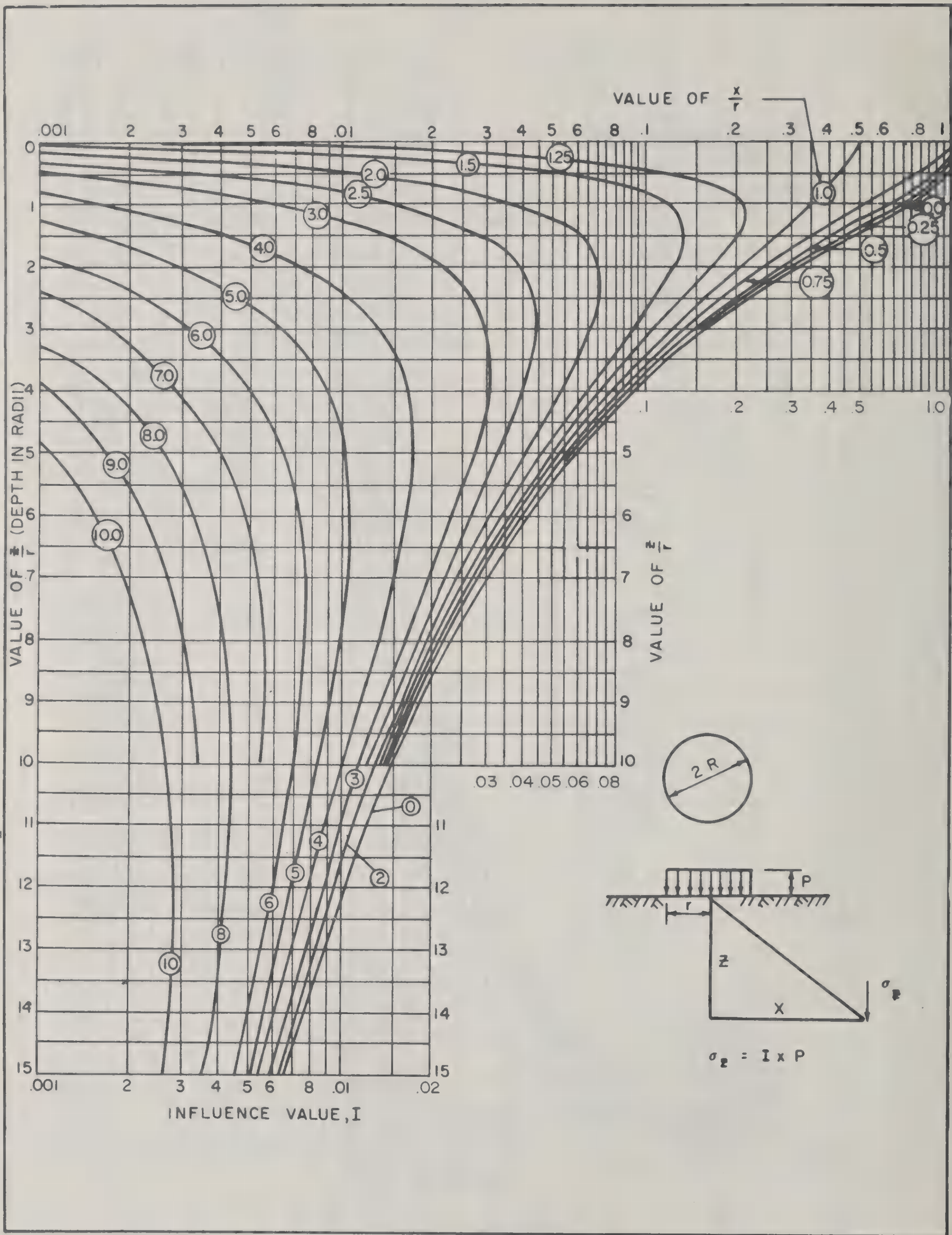


FIGURE 5-4
Influence Value for Vertical Stress Under Uniformly Loaded Circular Area

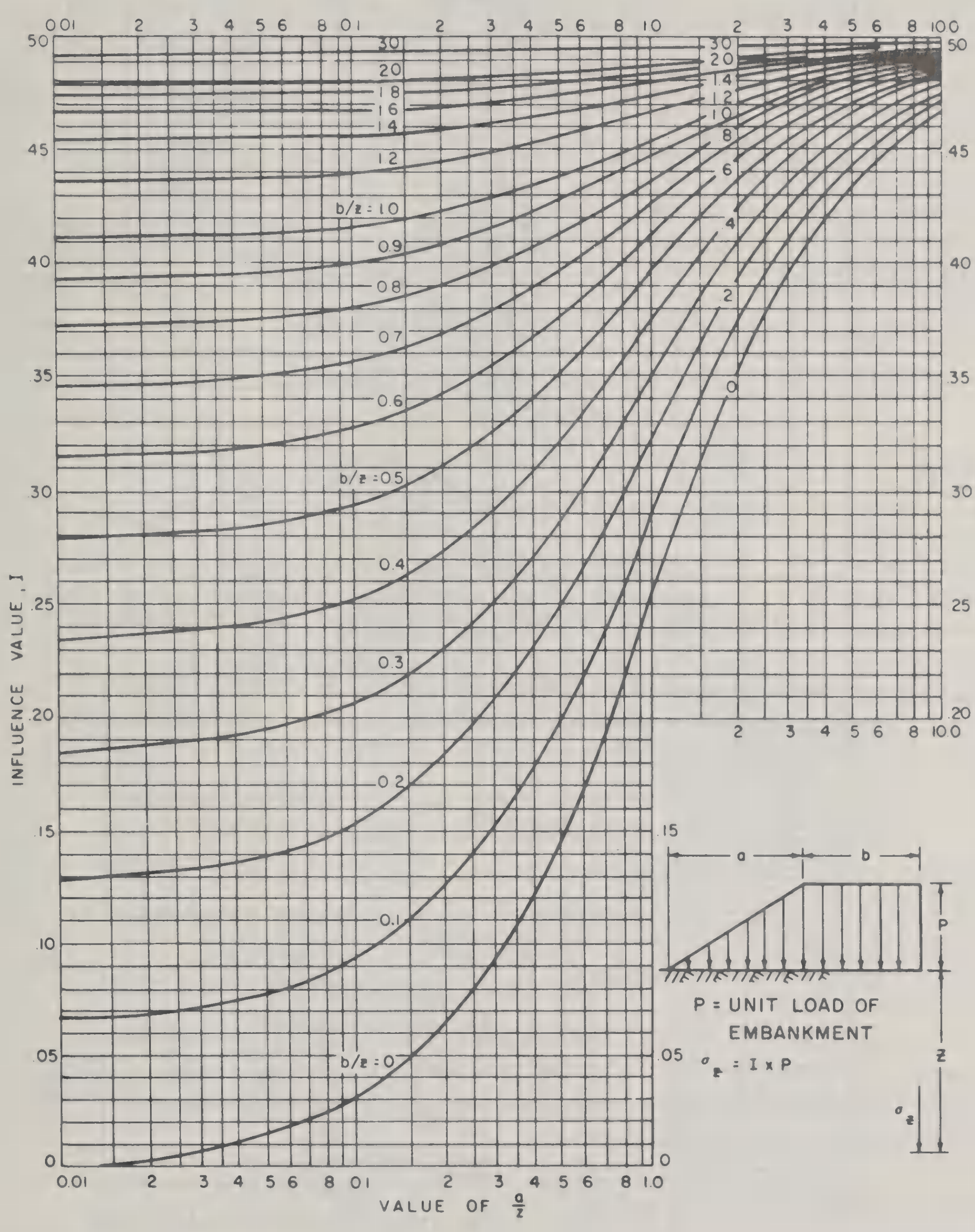


FIGURE 5-5
Influence Value for Vertical Stress Under Embankment Load of Infinite Length

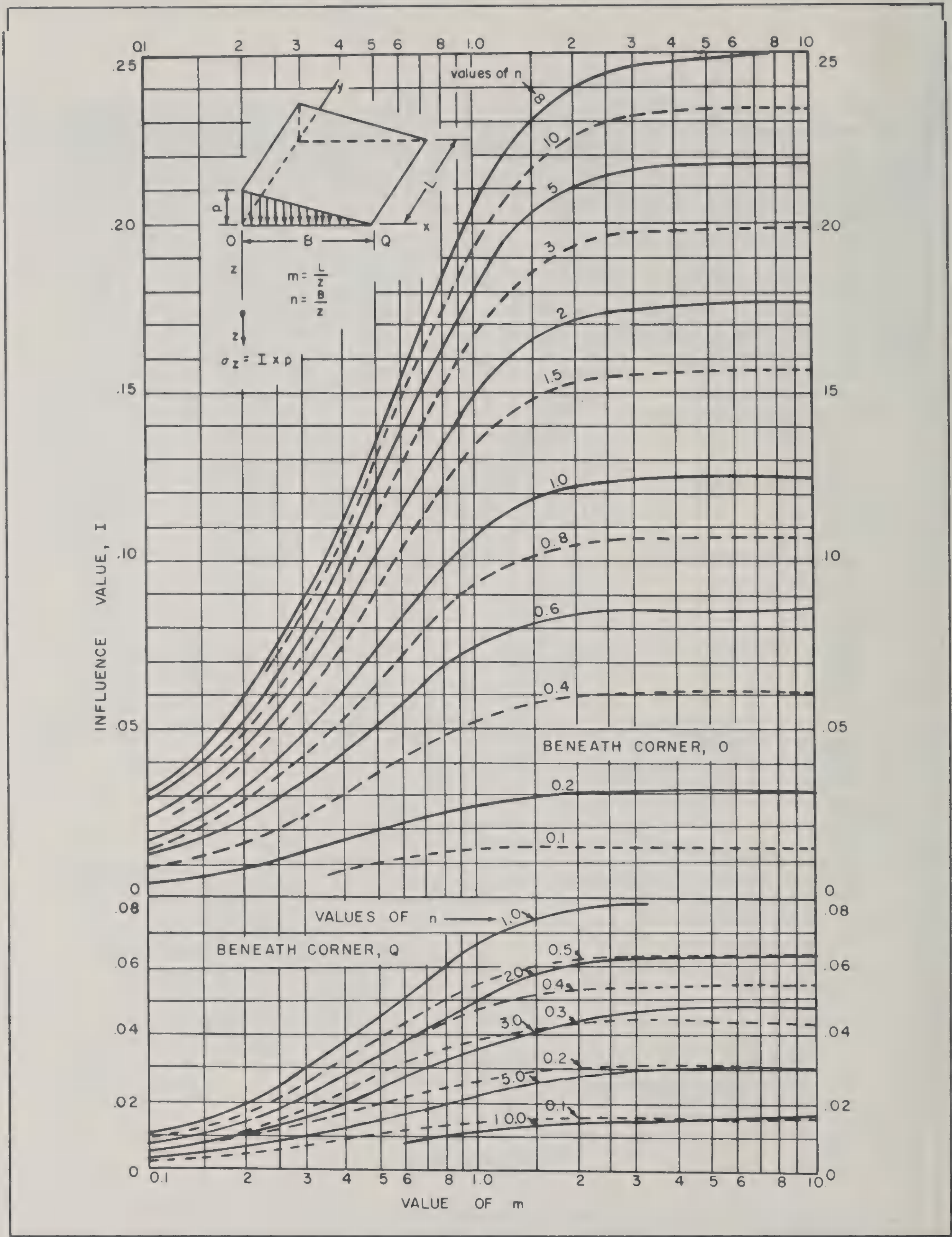
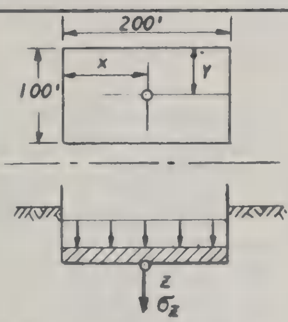


FIGURE 5-6
Influence Value for Vertical Stress Under Corners of Triangular Load of Finite Extent

RECTANGULAR MAT FOUNDATION



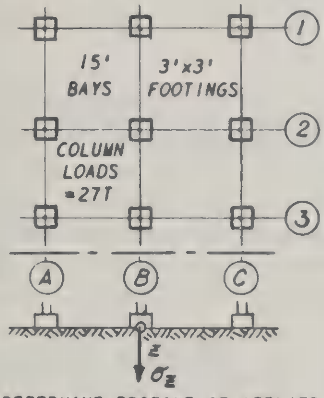
UNIT AREA LOAD (INCL. MAT) = 2TSF
UNIT LOAD EXCAVATED = .8TSF
NET AREA LOAD = 1.2TSF
DETERMINE PROFILE OF APPLIED STRESS BENEATH CENTER OF MAT

REFER TO FIGURE 5-3 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	0.120	0.480	0.576

SEPARATE COLUMN FOOTINGS



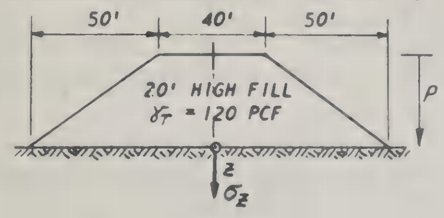
DETERMINE PROFILE OF APPLIED STRESS BENEATH B2.

REFER TO FIGURE 5-4 FOR INFLUENCE VALUES I FOR VERTICAL STRESS OUTSIDE OF UNIFORMLY LOADED CIRCULAR AREA. SQUARE FOOTING ASSUMED EQUIVALENT TO CIRCULAR AREA.

EQUIVALENT r:
$r = (\frac{9}{\pi})^{\frac{1}{2}} = 1.695'$
$p = 3.0\ TSF$
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	$\sigma_z = \Sigma Ip$ TSF
2	1.18	0.54	0	0	0.540	1.620
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

EMBANKMENT LOAD



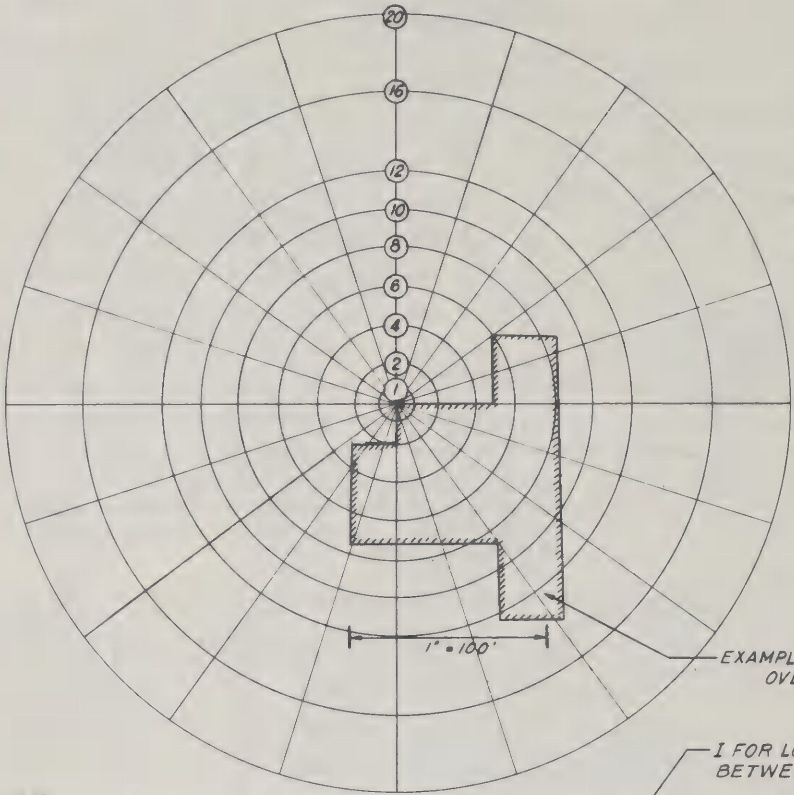
DETERMINE PROFILE OF APPLIED STRESS σ_z BENEATH ℄ OF EMBANKMENT OF INFINITE LENGTH.

REFER TO FIGURE 5-5 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	$\sigma_z = 2Ip$ 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

FIGURE 5-7
Examples of Computation of Vertical Stress



DEFINITIONS:

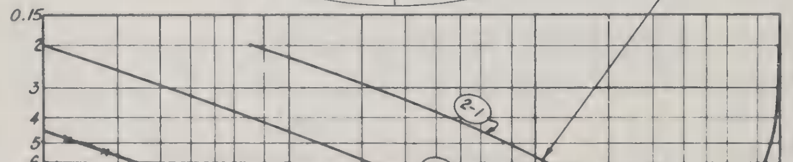
z = DEPTH TO POINT CONSIDERED

p = LOAD PER UNIT AREA

A = PROPORTION OF ANNULAR SPACE COVERED BY LOAD.

$\sigma_z = p \Sigma A I$

R = RADIUS OF CIRCLE NO. 1 (CONCENTRIC CIRCLES ARE SHOWN WITH RADII PROPORTIONAL TO 1, 2, 4, 6, 8, 10, 12, 16 AND 20).



EXAMPLE:
DETERMINE VERTICAL STRESS σ_z AT DEPTH OF 24 FT BENEATH CORNER OF LOAD SHOWN ABOVE:

p = 2.0 TSF

z = 24 FT

SCALE: 1" = 100'

(20)R = 200'

R = 10'

$\frac{z}{R} = \frac{24}{10} = 2.4$

RADII	A	I	A × I
16 - 12	0.018	0.0048	0.0001
12 - 10	0.060	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
			Σ = 0.2831

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

FIGURE 5-8
Influence Chart for Vertical Stress Beneath Irregular Load

2. LAYERED OR ANISOTROPIC FOUNDATIONS. Actual foundation materials differ from the isotropic, semi-infinite mass in two principal respects: (a) the modulus of elasticity may differ from layer to layer, the extreme condition being a soft surface stratum on a rigid, unyielding base such as bedrock; (b) a sediment is frequently more rigid in a horizontal than a vertical direction.

a. Vertical Stresses in Layered Foundations. For a general study of stresses in layered foundations beneath a circular loaded area see Mehta and Veletsos, *Stresses and Displacements in Layered Systems* (Bibliography). See Figure 5-9 for examples of vertical stresses in a two-layer foundation with various ratios of modulus of elasticity.

(1) Rigid Surface Layer. If the surface layer is the more rigid it acts as a distributing mat and vertical stresses directly beneath the load are less than Boussinesq values.

(2) Less Rigid Surface Layer. If the surface layer is the less rigid, vertical stresses in the upper layer exceed Boussinesq values. For influence diagrams for vertical stresses beneath rectangular loaded areas, see Burmister, *Stress and Displacement Characteristics of a Two-Layer Rigid Base Soil System* (Bibliography). Use these influence diagrams to determine vertical stress distribution for settlement analysis involving a soft surface layer underlain by stiff material.

b. Influence of Horizontal Rigidity. If subsoil has greater horizontal than vertical rigidity, vertical stresses beneath the center of load are less than Boussinesq values. Usually this effect is not as significant as the difference in rigidity from layer to layer.

c. Critical Depth. This is the depth below the foundation bearing level within which soil compression contributes significantly to surface settlements. In fine grained compressible soils, critical depth extends to that point where applied stress decreases to about 10 percent of overburden pressure. In coarse grained material critical depth extends to that point where applied stress decreases to about 20 percent of overburden pressure. As an approximate guide for settlement analysis, compute vertical stresses and compression of subsoils down to the critical depth. If there is no distinct change in the character of subsurface strata within this depth, elastic solutions for layered foundations need not be considered.

Section 4. PRINCIPAL STRESSES AT ACTIVE OR PASSIVE STATE

1. CONDITIONS. In the case of retaining walls, bulkheads, buried anchorages, or slope stability, sufficient movement may occur to produce active or passive conditions in the surrounding soil.

a. Active State. Where a soil mass stretches horizontally enough to fully mobilize shear strength, a condition of plastic equilibrium is reached; this is designated as the active state of stress. Active pressures are minor principal stresses where body forces tend to produce movement. The ratio of horizontal active pressures to the vertical stress of body forces is the active coefficient K_a in cohesionless soil.

b. Passive State. Where a soil mass is compressed horizontally, mobilizing shear resistance, a passive state of stress is ultimately reached. Passive pressures are major principal stresses where body forces tend to resist failure. The ratio of horizontal passive stresses to the vertical stress of body forces is the passive coefficient K_p in cohesionless soil.

2. COMPUTATION OF ACTIVE AND PASSIVE PRESSURES. See Figure 5-10 for formulas for active and passive pressures on a frictionless vertical face with horizontal ground surface. Three basic conditions required for validity of the formulas are listed in Figure 5-10. Under these conditions the failure surface is a straight plane and the formulas represent pressures required for equilibrium of the wedged shaped failure mass.

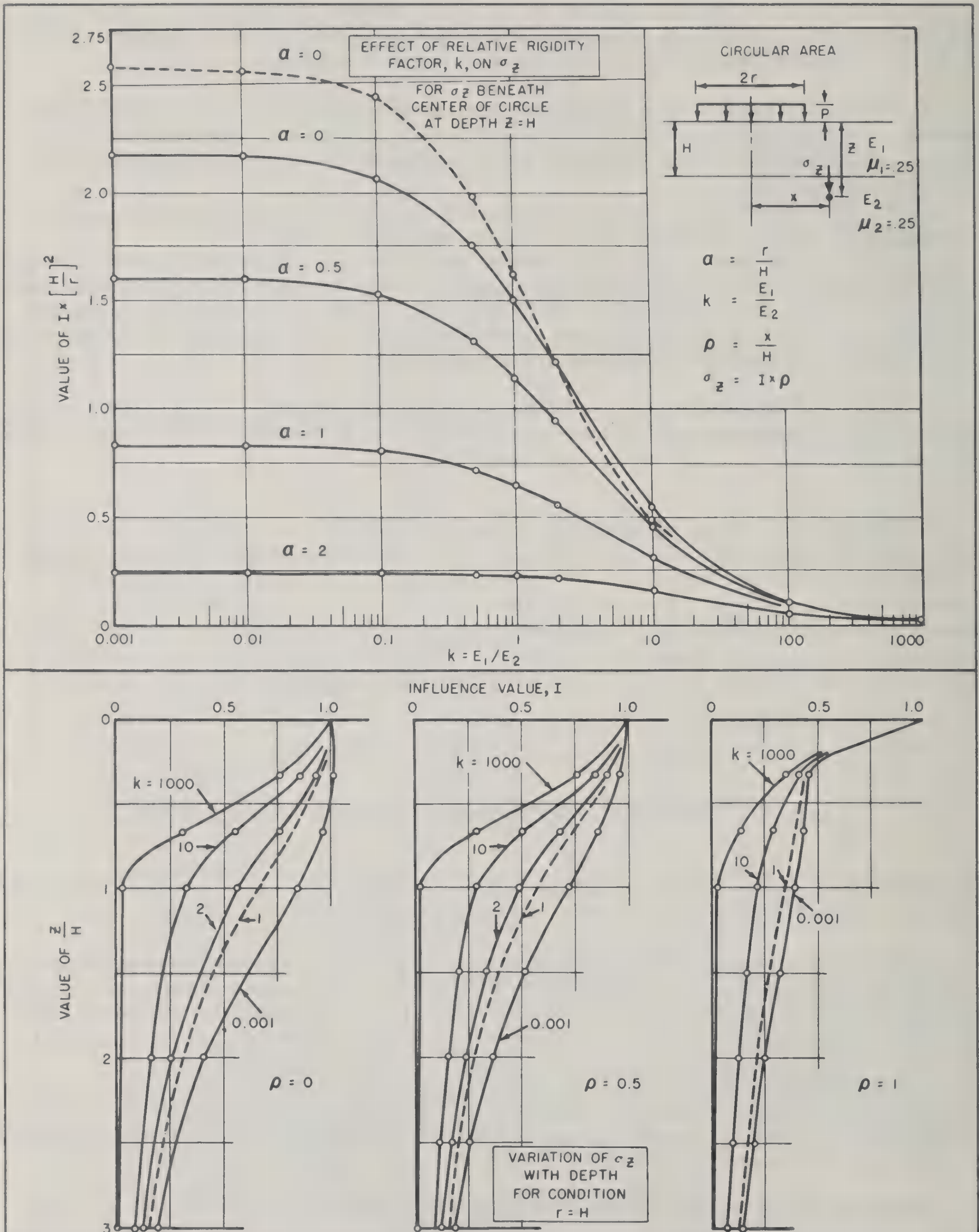
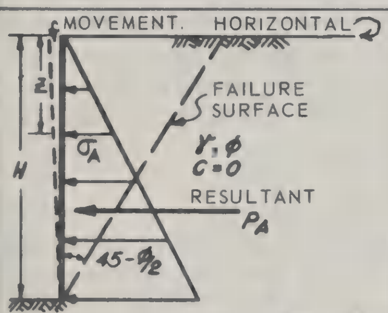


FIGURE 5-9
Influence Values for Vertical Stresses Beneath Uniformly Loaded Circular Area (Two-Layer Foundation)

COHESIONLESS SOIL	COHESIVE SOIL, NO FRICTIONAL RESISTANCE	COMBINED COHESION AND FRICTION

ACTIVE PRESSURES

Cohesionless soil

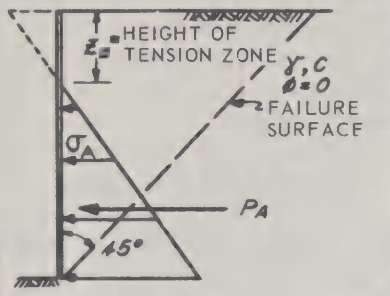


$K_A = \tan^2(45-\phi/2)$

$\sigma_A = K_A \gamma Z$

$P_A = K_A \gamma H^2/2$

Cohesive soil, no frictional resistance

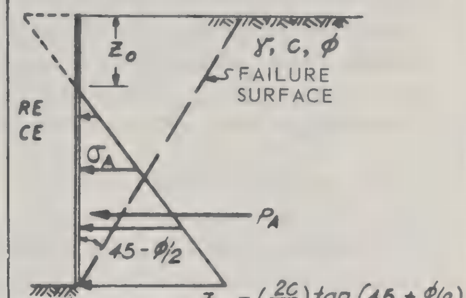


$Z_o = 2c/\gamma$

$\sigma_A = \gamma Z - 2c$

$P_A = \gamma H^2/2 - 2cH + \frac{2c^2}{\gamma}$

Combined cohesion and friction



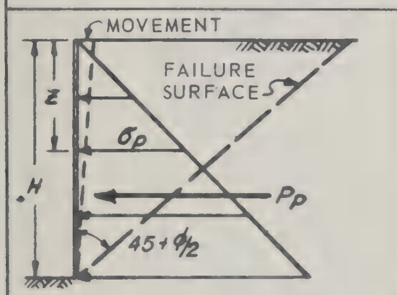
$Z_o = (\frac{2c}{\gamma})\tan(45+\phi/2)$

$\sigma_A = \gamma z \tan^2(45-\phi/2) - 2c\tan(45-\phi/2)$

$P_A = (\frac{\gamma H^2}{2})\tan^2(45-\phi/2) - 2cH\tan(45-\phi/2) + \frac{2c^2}{\gamma}$

PASSIVE PRESSURES

Cohesionless soil

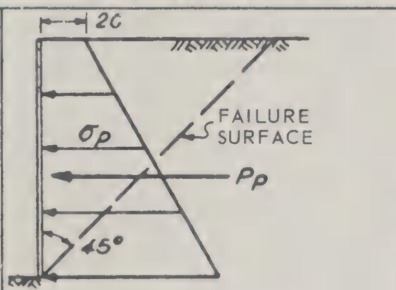


$K_p = \tan^2(45+\phi/2)$

$\sigma_p = K_p \gamma Z$

$P_p = K_p \gamma H^2/2$

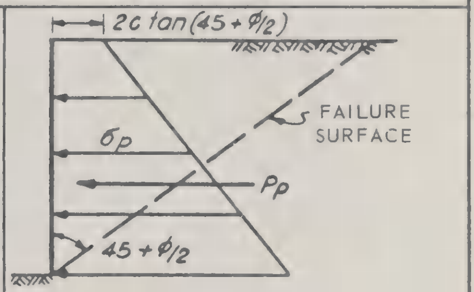
Cohesive soil, no frictional resistance



$\sigma_p = \gamma Z + 2c$

$P_p = \frac{1}{2}\gamma H^2 + 2cH$

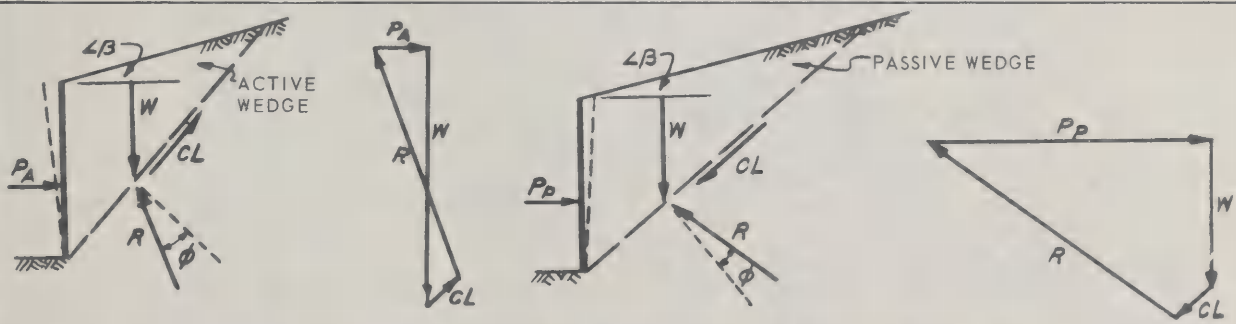
Combined cohesion and friction



$\sigma_p = \gamma z \tan^2(45+\phi/2) + 2c\tan(45+\phi/2)$

$P_p = (\frac{\gamma H^2}{2})\tan^2(45+\phi/2) + 2cH\tan(45+\phi/2)$

GRAPHIC SOLUTION FOR SLOPING BACKFILL



For cohesionless soils with sloping backfill, values of K_A and K_p, and positions of failure surface are given in Figure 5-12.. For soil with c and ϕ, the position of the failure surface is determined by analyzing trial wedges to obtain maximum value of P_A and minimum value of P_p.

The cases shown involve the following assumptions.

1. Materials are homogeneous.
2. Sufficient movement has occurred so shear strength on rupture surface is completely mobilized.
3. Wall is vertical. No shear forces are present on back of wall. Resultant forces are horizontal.

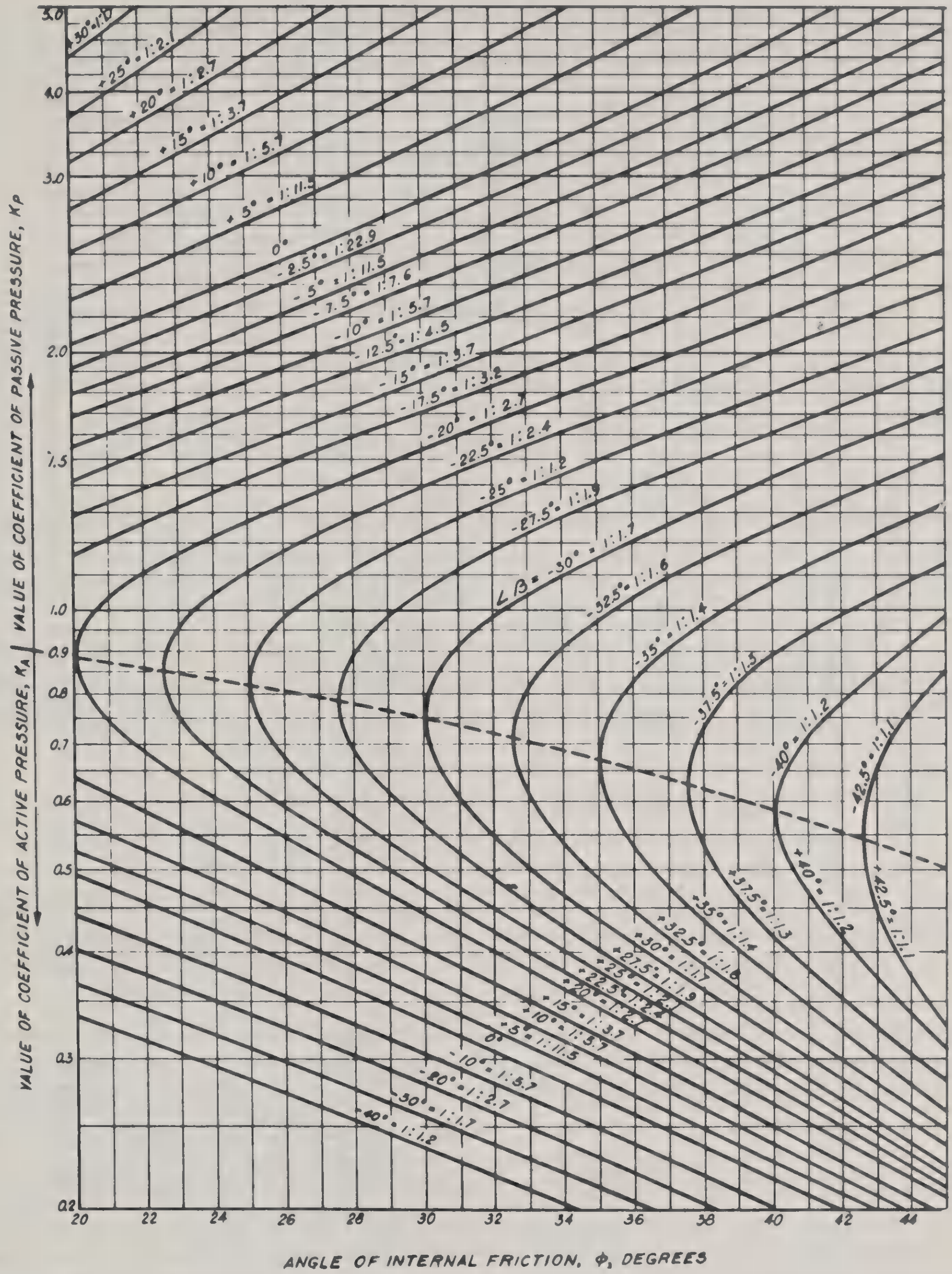
Under these conditions, resultant pressures are active and passive values, and rupture surface is a straight plane through heel of wall. Effects of surcharge and ground water pressures are not included.

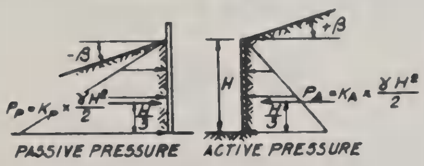
FIGURE 5-10
Computation of Simple Active and Passive Pressures

a. Straight-Slope Backfill. Compute active or passive pressures for cohesionless materials with straight-slope backfill from coefficient of Figure 5-11. For a straight-slope backfill in soil with both friction and cohesive strength, determine resultant active or passive force by analyzing equilibrium of the failure wedge, as shown in Figure 5-10. Obtain location of resultant active or passive force by summing moments of forces acting on the wedge about its toe. Distribute active or passive pressures on the wall to conform to this location of the resultant. See Figure 5-12 for position of the critical failure plane in a cohesionless mass with a straight-slope backfill.

b. Irregular Backfill. Compute active or passive pressures for cohesive soils with irregular backfill by successive trials to determine the critical failure wedge. Use location of failure plane given in Figure 5-12 as a first approximation of the trial wedge to be analyzed. Critical failure wedge is that one which requires maximum active pressures or minimum passive pressures for its equilibrium.

c. Additional Solutions. For analysis of pressures with more complicated conditions, including wall friction, ground water, and stratified soils, see Chapter 10.



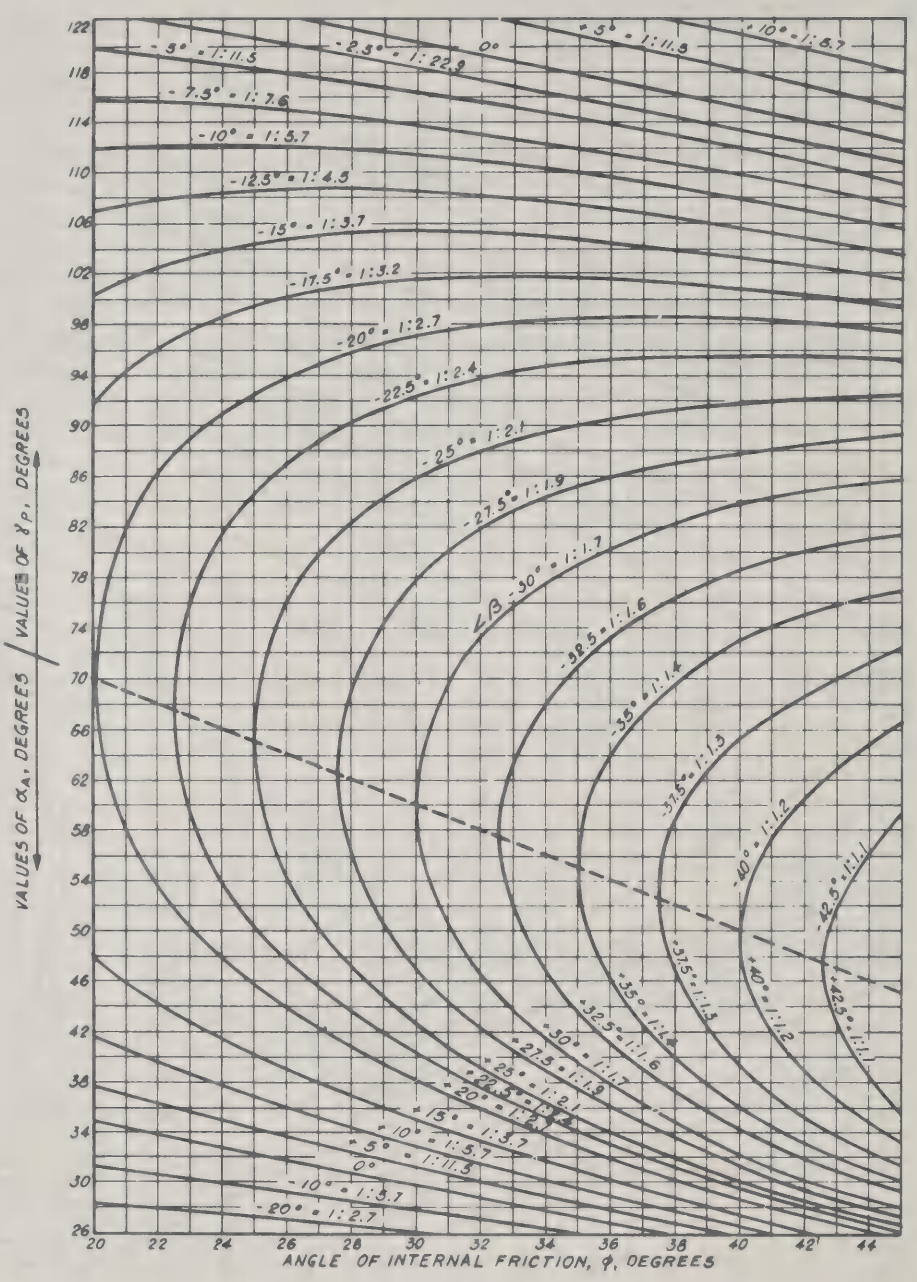


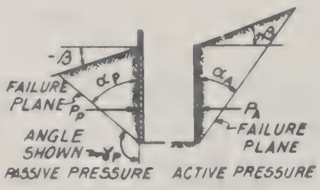
$K_A = \left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi - \cos\phi \tan\beta)}}\right]$; $K_P = \left[\frac{\cos\phi}{1-\sqrt{\sin\phi(\sin\phi - \cos\phi \tan\beta)}}\right]$

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
P_P = PASSIVE RESULTANT
γ = UNIT WEIGHT OF SOIL
H = HEIGHT OF WALL
ϕ = ANGLE OF INTERNAL FRICTION
β = SLOPE ANGLE

FIGURE 5-11
Active and Passive Coefficients, Sloping Backfill
(Cohesionless Soils)





$\cot \alpha_A = \tan\phi + \sqrt{1 + \tan^2\phi - \frac{\tan\beta}{\sin\phi\cos\phi}}$ $\quad$ $\cot \alpha_P = -\tan\phi + \sqrt{1 + \tan^2\phi - \frac{\tan\beta}{\sin\phi\cos\phi}}$

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 5-11.

FIGURE 5-12
Position of Failure Surface for Active and Passive Wedges (Cohesionless Soils)

CHAPTER 6. SETTLEMENT ANALYSIS

Section 1. INTRODUCTION

1. **SCOPE.** This chapter concerns (a) settlement produced by volume change during consolidation, (b) criteria for tolerable settlement, and (c) methods of reducing or accelerating consolidation. Procedures described apply to compressible fine grained strata subjected to loads of wide extent. For analysis of settlement of isolated footings from shear strain, see Chapter 11.

2. **PURPOSE.** For foundations on clay and silt, it is essential to make a reliable estimate of the probable differential settlement to determine if the design contemplated is satisfactory. If differential settlement will be excessive, alter the proposed foundation type or take measures to reduce settlements.

3. **MECHANICS OF CONSOLIDATION.** See Figure 6-1. Superposed 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.

Section 2. ANALYSIS OF STRESS CONDITIONS

1. **INITIAL STRESSES.** See Figure 6-2 for typical profiles of vertical stress existing in a compressible stratum prior to construction. For equilibrium conditions with no hydrostatic excess pressures, compute vertical effective stress from the effective unit weight of overburden. (Case (1), Figure 6-2.)

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 or excavation, (2) by capillary stresses from desiccation, and (3) by lower ground water 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 ground water or recent addition of fills or structural loads. Residual hydrostatic excess pore pressure existing in the compressible stratum will dissipate with time, causing future settlements.

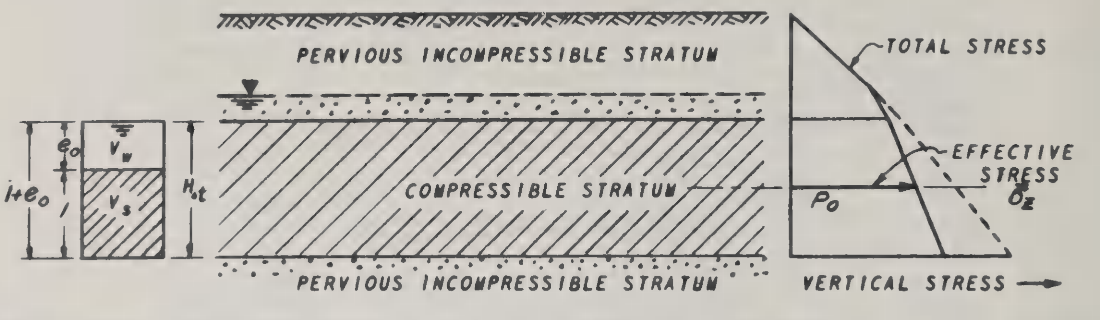
c. Evaluation of Existing Conditions. Determine consolidation condition at start of construction by the following steps:

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

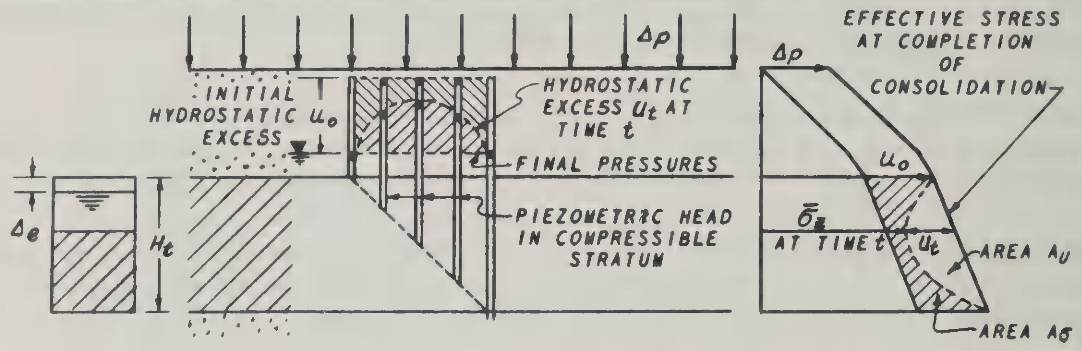
(2) Compare profile of preconsolidation stress determined from laboratory consolidation tests (Chapter 3) with the profile of effective overburden pressures.

(3) Estimate preconsolidation from s/p_c ratio using laboratory shear strength and check by the approximate relationship with the liquidity index. (Figure 3-1.)

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



CONDITIONS BEFORE LOADING



CONDITIONS DURING LOADING

COMPUTATION OF TOTAL SETTLEMENT

ΔH = TOTAL SETTLEMENT FROM PRIMARY CONSOLIDATION PLUS AN AMOUNT OF SECONDARY COMPRESSION.

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

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

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

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

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

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

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

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_o.

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

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

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 FIG. 6-6.

$T_v = \frac{C_v t}{H^2}$ $\Big\{$ 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 6-1
Consolidation Settlement Analysis

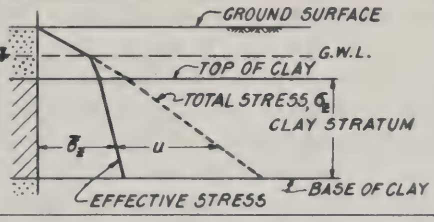
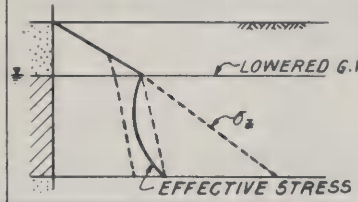
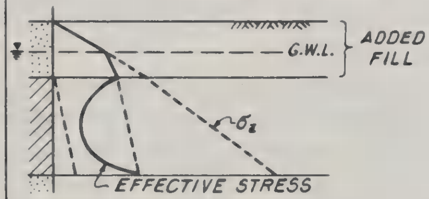
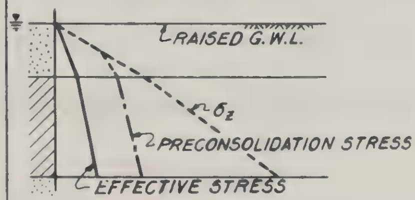
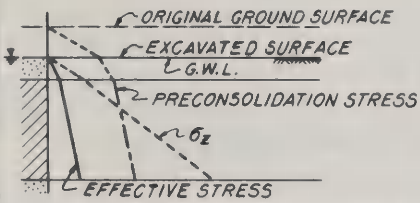
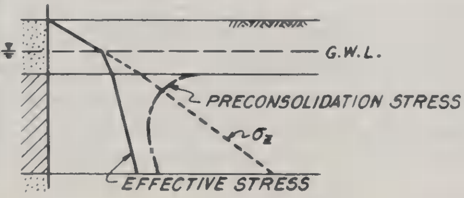
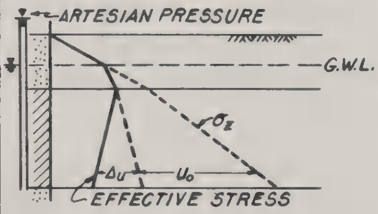
<table>
<tr><th></th><th>Stress condition</th><th>Diagram of vertical stresses</th><th>Description</th></tr>
<tr><td></td><td>(1)
Simple overburden pressure</td><td>
</td><td>Effective stress of overburden is computed directly from unit weights of soil, γ_T above GWL, γ_{sub} below GWL. Total stress equals effective stress plus neutral pressure of ground water:
$$\sigma_z = \bar{\sigma}_z + u$$</td></tr>
<tr><td rowspan="2">Hydrostatic excess pressure</td><td>(2)
Lowering of ground water level</td><td>
</td><td>Lowering of ground water increases effective stress as hydrostatic excess pore pressure dissipates and consolidation of clay takes place. Total stress practically unchanged.</td></tr>
<tr><td>(3)
Partial consolidation under weight of initial fill</td><td>
</td><td>Stresses caused by adding fill on top of clay layer. Effective stress moves to the right, approaching effective pressure of overburden as consolidation proceeds.</td></tr>
<tr><td rowspan="3">Preconsolidated conditions</td><td>(4)
Rise of ground water level</td><td>
</td><td>Rise of ground water level decreases effective pressure of overburden. Effective stress line moves to left. Then preconsolidation stress equals original effective stress of overburden. Total stress practically unchanged.</td></tr>
<tr><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></td><td>(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 6-2
Profiles of Vertical Stresses Before Construction

d. Computation of Added Stresses. In general, use the elastic solutions (Chapter 5) to determine the vertical stress increment from applied loads. On vertical lines beneath selected points in the loaded area, plot profiles of estimated preconsolidation, effective overburden stress plus the increment of applied stress. See Figure 6-3 for typical profiles. For rigid boxes, piers, or silos, apply formulas for stress distribution beneath rigid structures. Lowering of ground water during construction or regional drawdown reduces water pressures 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. SETTLEMENT COMPUTATION

1. TOTAL SETTLEMENT. If preconsolidation stress is determined reliably, total settlement may be predicted with reasonable accuracy. The percentage error is greatest for settlement from recompression only. In this case an overestimate of several hundred percent may result unless high quality undisturbed samples are utilized for consolidation tests.

a. Typical Loading Cycle. See Figure 6-3 for loading sequence in building construction. Foundation excavation causes swell. Application of a structural load recompresses subsoil and may extend consolidation into a virgin compression range. Stress changes are plotted on a semilogarithmic pressure-void ratio e-p curve in Figure 6-3.

b. Pressure-Void Ratio Diagram. Determine the appropriate e-p curve to represent average properties of compressible stratum from consolidation tests. The e-p curve may be interpreted from straight line virgin compression and recompression slopes intersecting at the preconsolidation stress. Draw e-p curve to conform to these straight lines as shown in Figure 6-3. Depending on variability of material, more than one e-p curve representing different sections of the stratum may be required.

c. Settlement Magnitude. Compute settlement magnitude from change in void ratio corresponding to change in stress from initial to final conditions, obtained from the e-p curve (Figure 6-3). A laboratory e-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.

2. DIFFERENTIAL SETTLEMENT. 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 slope of the settlement profile or the greatest difference in settlement between adjacent foundation units.

a. Approximate Values. In general, the ratio between settlement of corner, or edge, to settlement of the center of a structure depends on the flexibility of the structure, shape of loaded area, and thickness of compressible stratum. For simple, flexible structures, compute center settlement as described above and estimated edge or corner settlement from the elastic analysis summarized in the bottom panel of Figure 6-4. For a flexible square load on a thick stratum, corner settlement is one-half, and settlement at the center of a side is two-thirds, of center settlement. For the same load on a thin stratum, corner settlement is one-quarter, and settlement at the center of a side is one-half, of center settlement.

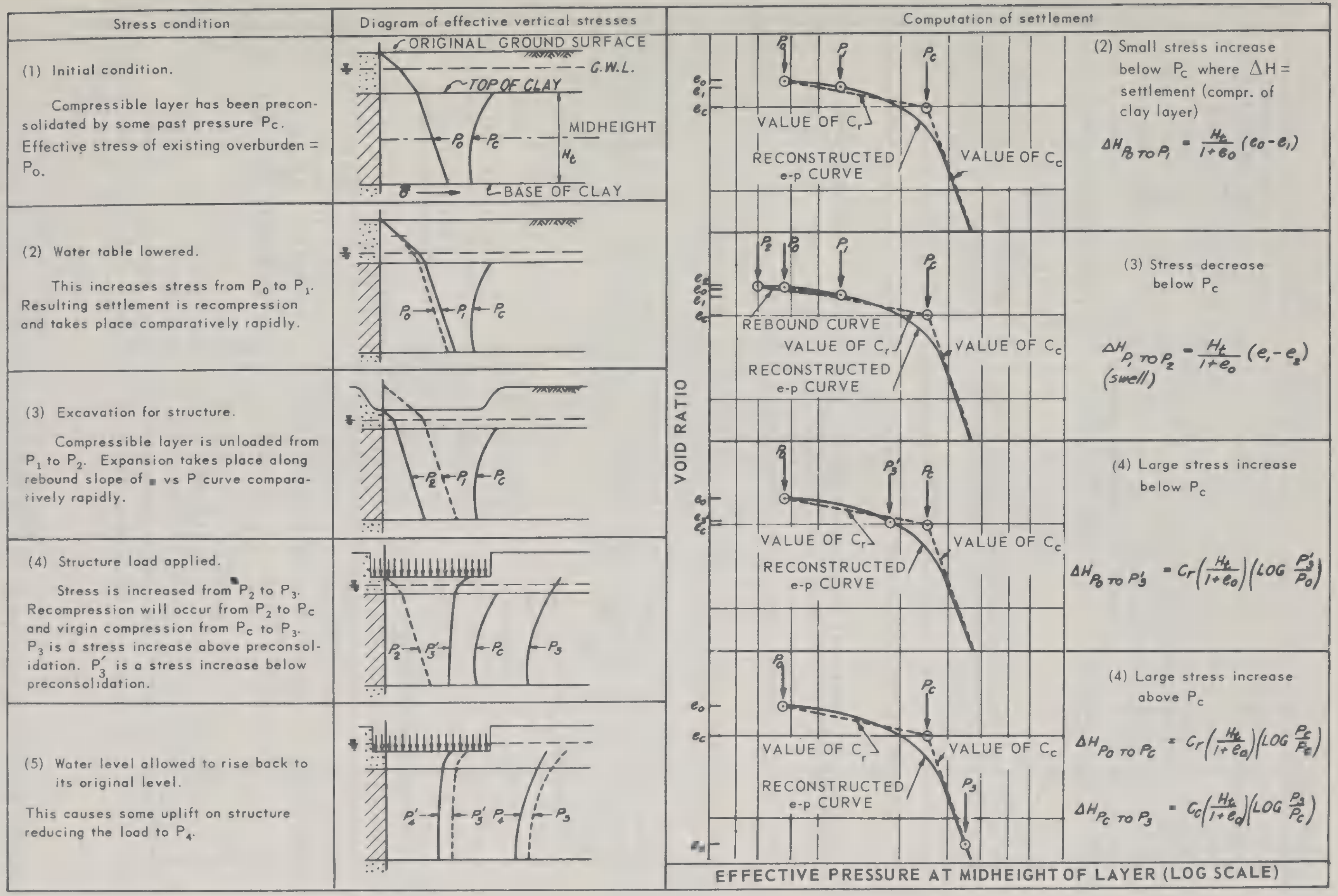


FIGURE 6-3
Computation of Total Settlement for Various Loadina Conditions

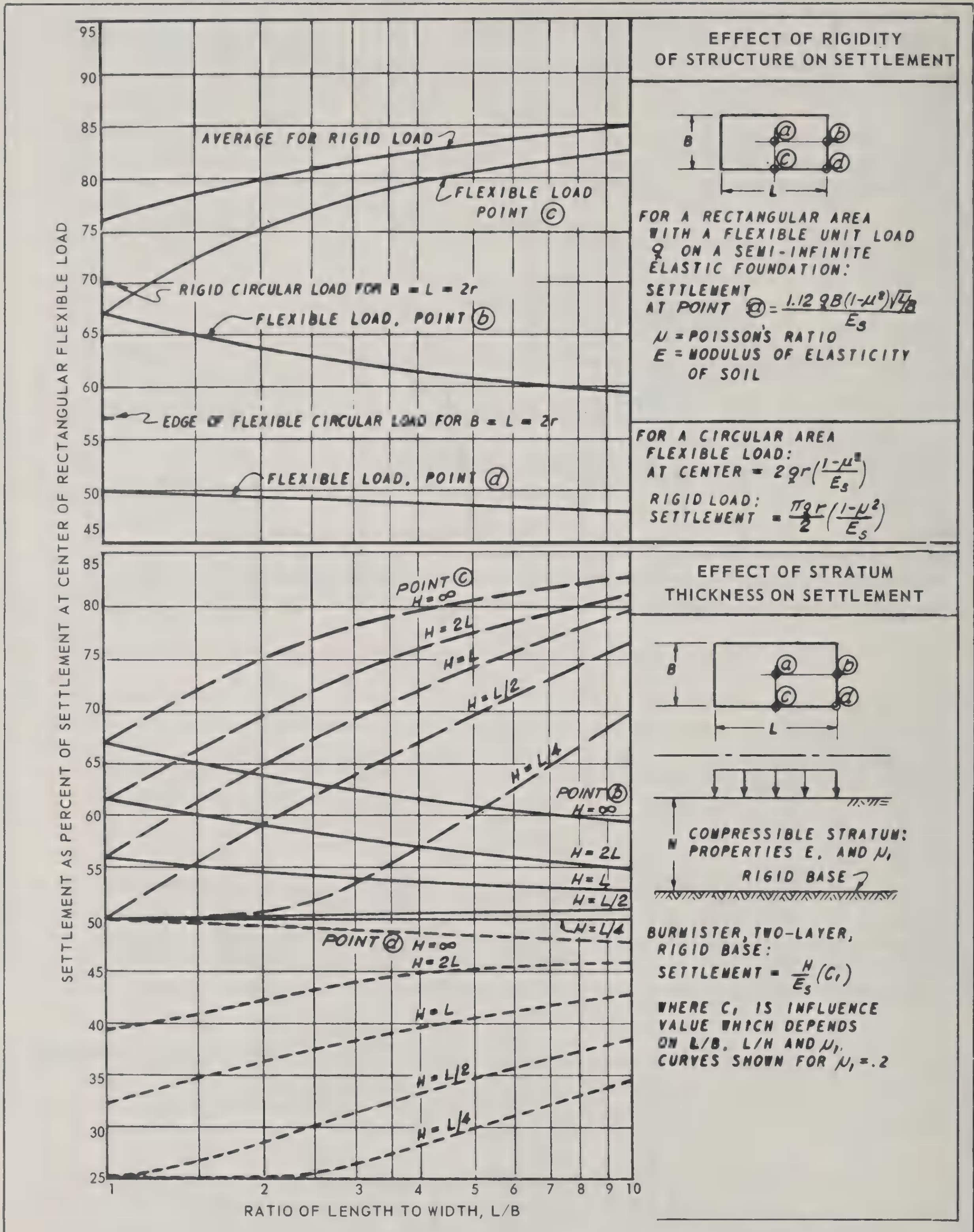


FIGURE 6-4
Approximate Effect of Rigidity of Structure and Compressible Stratum Thickness on Settlement

b. Effect of Structure Rigidity. Computed differential settlement is less accurate than computed total or average settlement because of the influence of the structure rigidity in redistributing loads on the foundation. A rigorous analysis of the effect of rigidity on settlement is generally not practicable. Assume a possible redistribution of loads from rigidity and compute settlements for this altered loading. Redistribution of loads may be based on diagrams of foundation contact pressures given in Terzaghi, *Theoretical Soil Mechanics* (Bibliography). For preliminary analysis, use the upper panel of Figure 6-4 to estimate the effect of structure rigidity on settlement. This diagram shows average settlement for a rigid load and settlement on the periphery of a flexible load for various shapes of loaded area. The rigidity of an ordinary multistory building frame reduces center settlement by about 6 to 12 percent of the computed value for flexible loading, and increases edge settlement by about 4 to 8 percent of the computed flexible value.

3. TOLERABLE SETTLEMENT OF STRUCTURES. Except where a structure is connected to a supported utility or adjacent building, differential settlement within the structure, rather than total settlement, causes damage. See Table 6-1 for approximate values of differential settlement of various structures that can occur without damage to frame or finish.

a. Structural Criteria. Where maximum differential settlements are expected to approach the tolerable values in Table 6-1, the designer should consider their effect on stresses in the structural frame.

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 will 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.

Section 4. MAGNITUDE OF SWELL

1. CAUSE. 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 6-2.

a. Reduction of Overburden Stresses. Compute resulting swell from laboratory e-p curve (Case (3) Figure 6-3) as follows:

(1) For this purpose, load the consolidation sample to the original pressure acting in situ and reduce pressure by the decrement of stress that will occur as a result of construction.

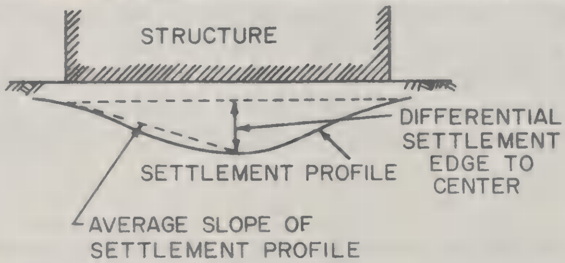
(2) For preliminary estimates, use values of swelling index or swelling ratio given in Figure 3-4.

(3) If time of unloading is short, total rebound may not occur. Estimate time rate of swell from laboratory consolidation characteristics to determine that portion of rebound which will be completed in the unloading period.

b. Changes in Capillary Stresses. An increase in pore water pressures from capillary tension to atmospheric pressure will result in a corresponding decrease in effective stress in grain skeleton and consequent swell as follows:

TABLE 6-1
Tolerable Differential Settlements of Structures

Type of structure	Tolerable differential settlement	Qualifying conditions
Circular steel petroleum or fluid storage tanks: Fixed top: Floating top:	(Units of radians of slope of settlement profile) 0.008 0.002 to 0.003 (depending on details of floating top).	Values apply to tanks on flexible base. Rigid slabs for base will not permit such settlement without cracking and local buckling.
Tracks for overhead traveling crane.	0.003	Value taken longitudinally along track. Settlement between tracks generally does not control.
Rigid circular mat or ring footing for tall and slender rigid structures such as stacks, silos, or water tanks.	0.002 (cross slope of rigid foundation)	
Jointed rigid concrete pressure pipe conduit.	0.015 (radians of angle change at joint)	Maximum angle change at joint is generally 2 to 4 times average slope of settlement profile. Damage to joint also depends on longitudinal extension.
One- or two-story steel frame, truss roof, warehouse with flexible siding.	0.006 to 0.008	Presence of overhead crane, utility lines, or operation of forklifts on warehouse floor would limit tolerable settlement.
One- or two-story houses with plain brick bearing walls and light structural frame.	0.002 to 0.003	Larger value is tolerable if significant portion of settlement occurs before interior finish is complete.
Structures with sensitive interior or exterior finish such as plaster, ornamental stone, or tile facing.	0.001 to 0.002	Larger value is tolerable if significant portion of settlement occurs before finish is complete.
Structures with relatively insensitive interior or exterior finish such as dry wall, movable panels, glass panels.	0.002 to 0.003	Damage to structural frame may limit tolerable settlements.
Multistory heavy concrete rigid frame on structural mat foundation 4 ft ± thick.	0.0015	Damage to interior or exterior finish may limit tolerable settlements.



Tolerable differential settlement is expressed in terms of slope of settlement profile

Value of 0.001 = 1/4-in. differential settlement in 20-ft distance

Value of 0.008 = 2-in. differential (settlement in 20-ft distance)

TABLE 6-2
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 ground water 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 ground water, 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 6-2 (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. Ground water 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.

(1) Where highly preconsolidated plastic soils are present at the ground surface, seasonal cycles of rainfall and desiccation produce volume changes. For three conditions in different climates, see Table 6-2.

(2) Climatic conditions have more influence on swell than clay properties alone. In an appropriate climate, any surface clay of medium to high plasticity with relatively low moisture content can heave.

(3) Estimate potential heave from consolidation test by measuring swell for different applied pressures after samples have been saturated and negative capillary stresses released. Choose applied pressures to simulate in situ effective stresses without capillary effect. Samples for the swelling test must be recovered at a time when capillary stresses are effective in situ. Samples obtained in the wet season may be in equilibrium with ordinary overburden stresses. See Figure 6-5 for method of computing total swell.

(4) Total swell beneath a foundation may be reduced to a tolerable value by excavation of a portion of the swelling clay and replacement by a nonswelling compacted fill. See Figure 6-5 for method of determining undercut required for this purpose.

Section 5. TIME RATE OF SETTLEMENT

1. APPLICATIONS. Settlement time rate must be determined for foundation treatment involving either acceleration of consolidation or preconsolidation before placing the final 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.

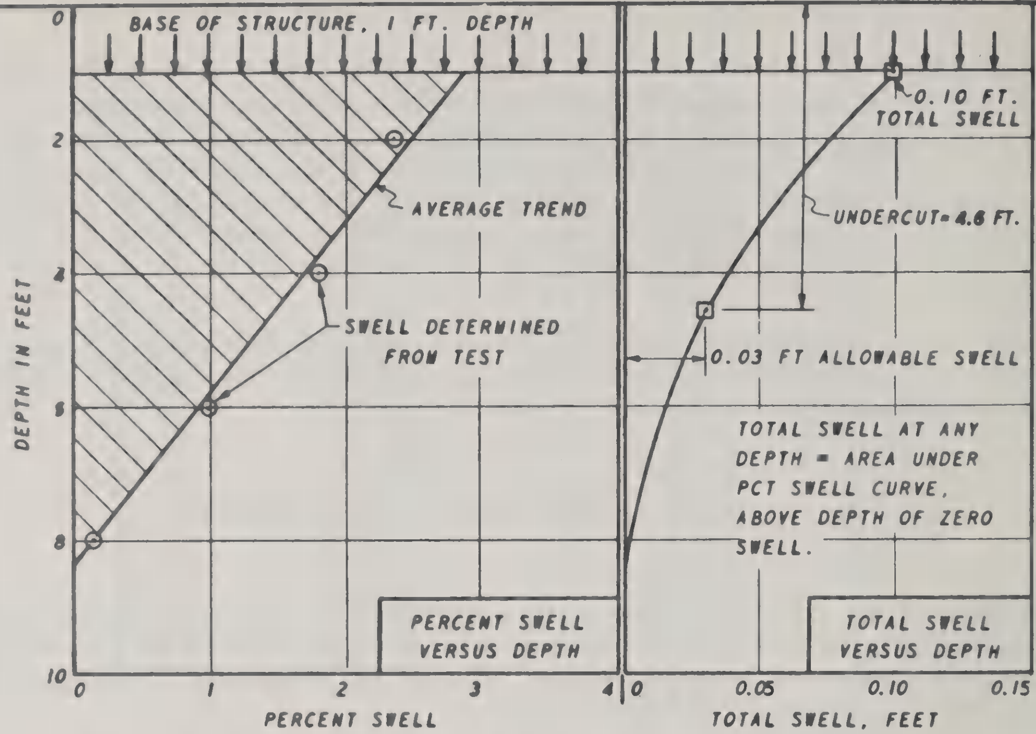
2. PRIMARY CONSOLIDATION. Where pore water drainage is essentially vertical, the ordinary theory of consolidation defines time rate of settlement. Using the coefficient of consolidation c_v, compute percent consolidation completed at specific elapsed times by the time factor T_v curves of Figure 6-6. See examples of computation in Figure 6-7.

a. Effect of Pressure Distribution. Rate of consolidation is influenced by the distribution of applied pressures on a vertical line. Use the top panel of Figure 6-6 for cases where applied pressures are constant or vary linearly with depth. For time factor curves for nonlinear pressure distribution, use Terzaghi and Peck, *Soil Mechanics in Engineering Practice*. For convenience, use nomograph of Figure 6-8 for consolidation rate under applied stresses that are constant with depth for single drainage, or are constant or vary linearly with depth for double drainage. For preliminary estimates, use values for the coefficient of consolidation given in Figure 3-5.

b. Accuracy of Prediction. Theoretical consolidation conditions frequently do not hold in situ because of horizontal drainage directed away from the loaded area. Furthermore, the laboratory coefficient of consolidation decreases drastically with sample disturbance, see Figure 3-5. Except where drainage is controlled, as in sand drain installations, predicted settlement time rate tends to be slower than actual rate.

c. Gradual Load Application. If construction time is appreciable compared to time required for primary consolidation, use the time factor curves of the middle panel of Figure 6-6 to determine consolidation rate during and following construction.

d. Consolidation of Two-Layer System. If a compressible stratum contains two layers of drastically different overall properties, use the procedure of Figure 6-9 to determine overall settlement time rate.



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 6-5
Computation of Swell of Desiccated Clays

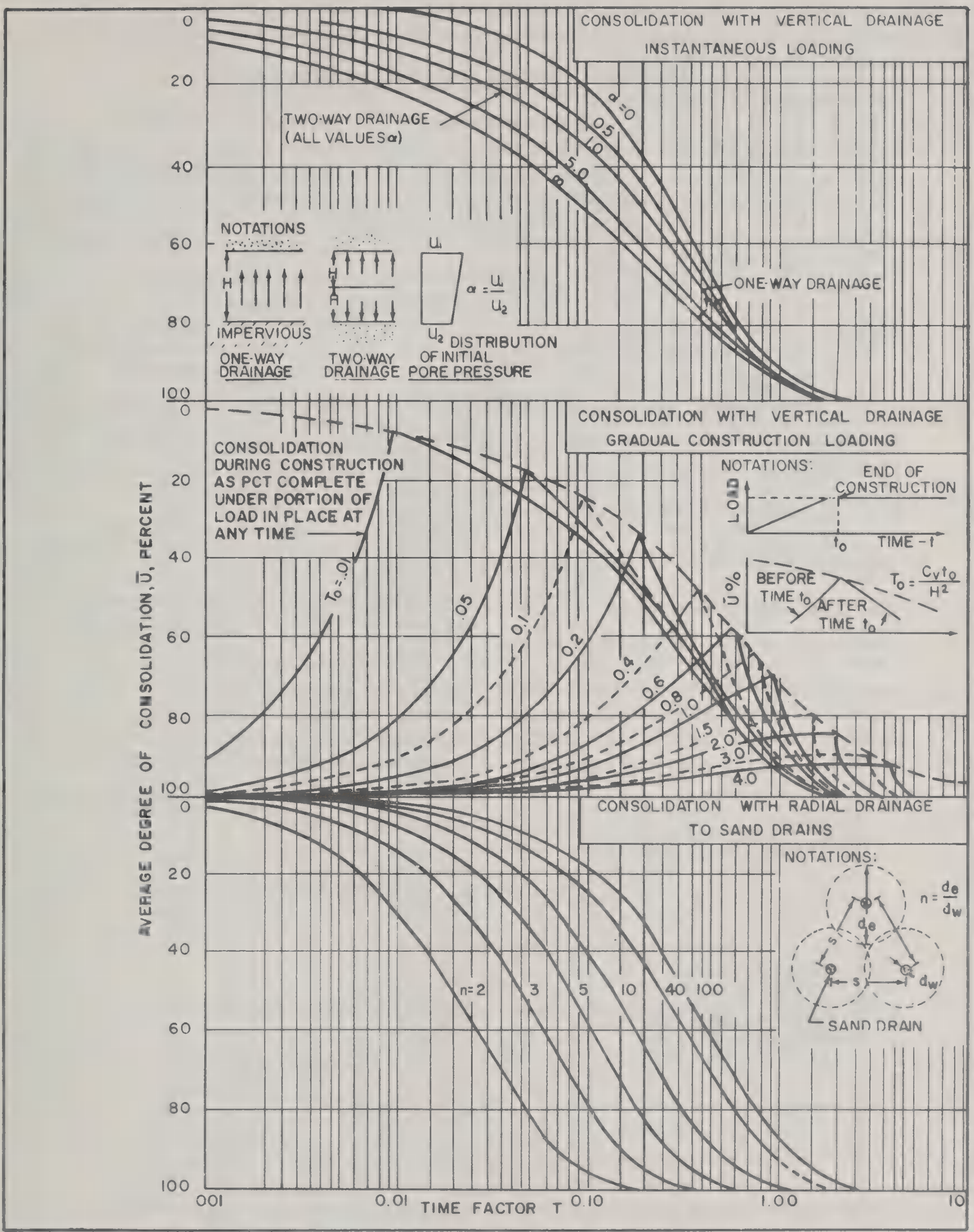


FIGURE 6-6
Time Factors for Consolidation Analysis

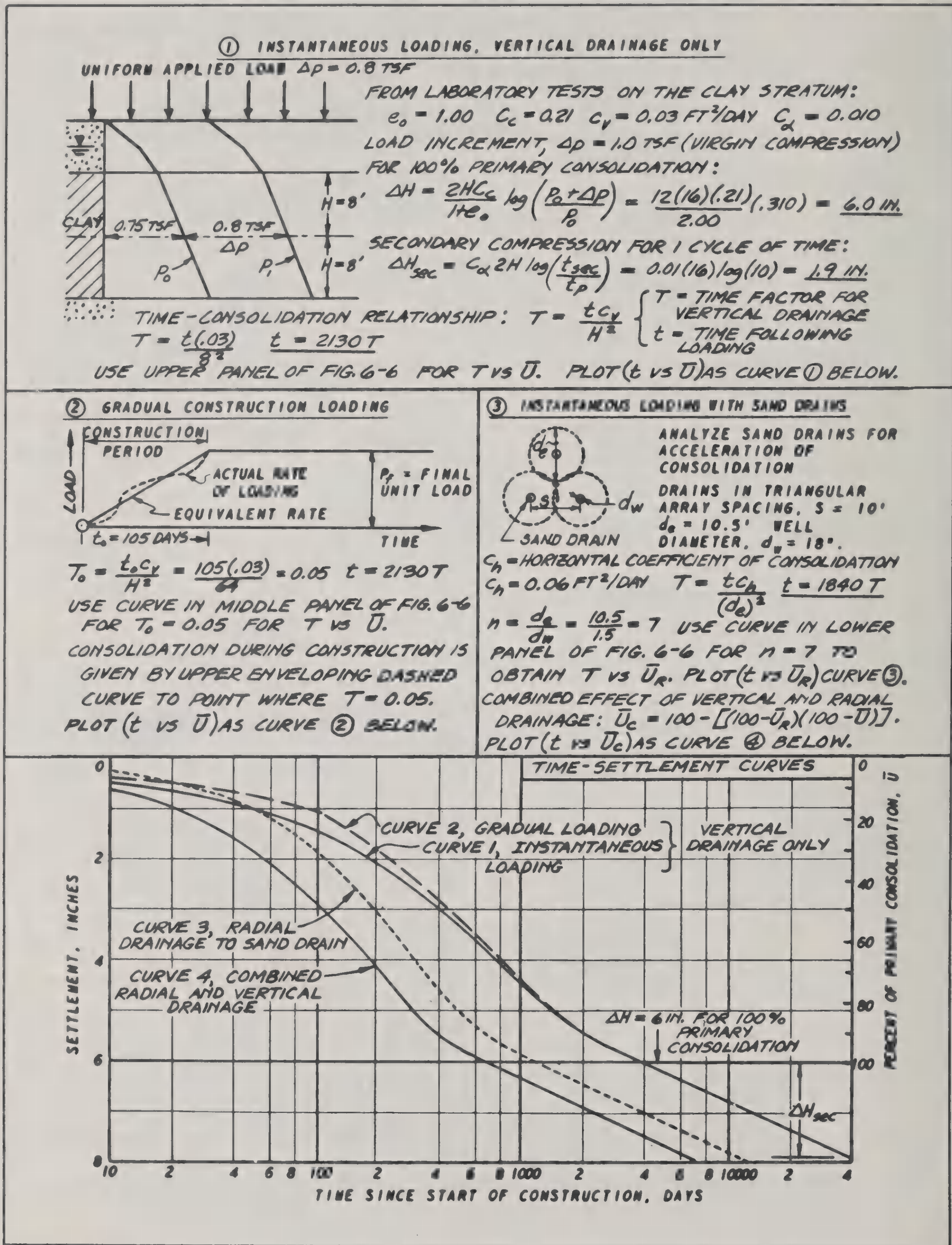


FIGURE 6-7
Example of Computation of Settlement Time Rate

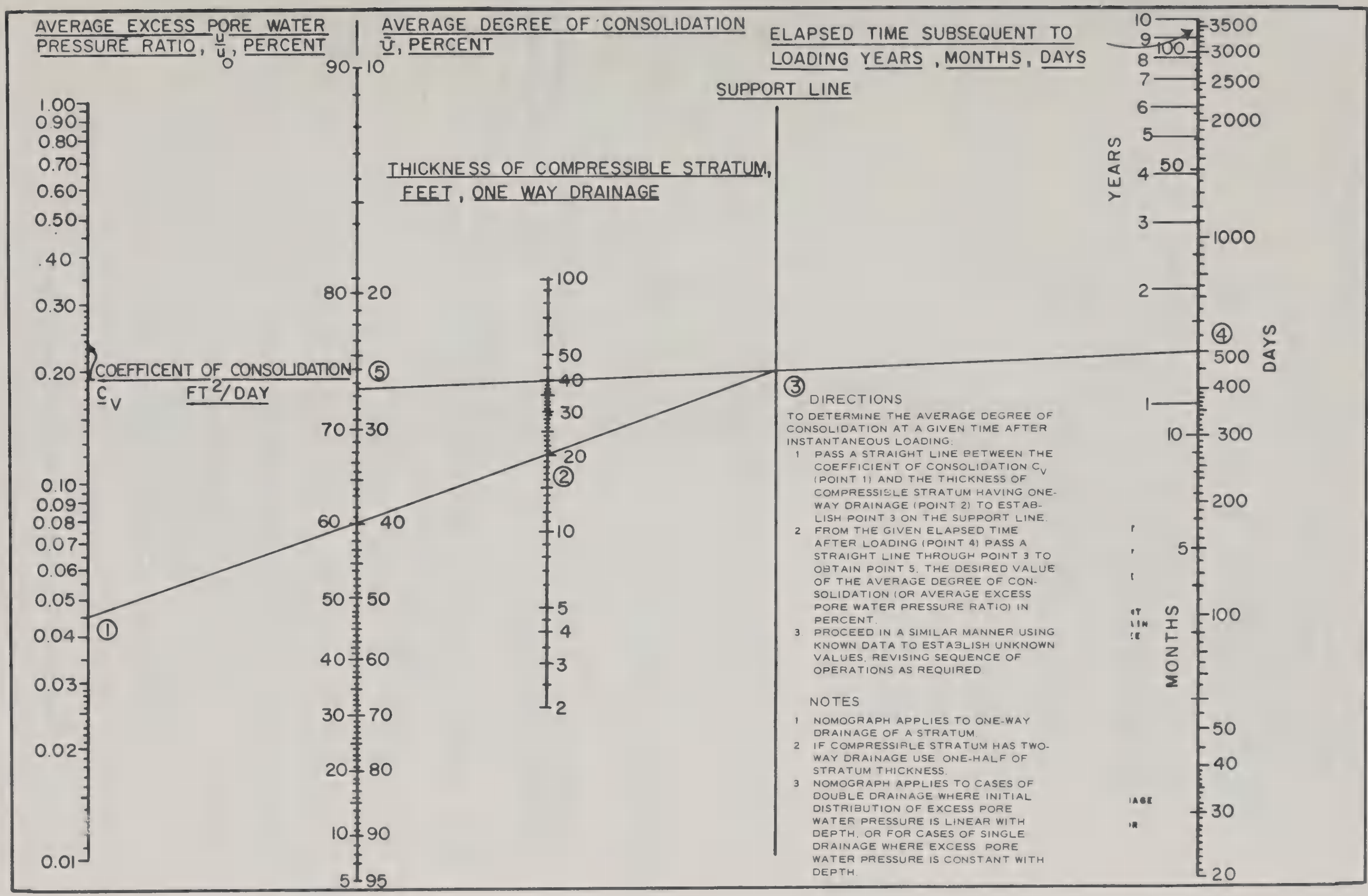


FIGURE 6-8
Nomograph for Consolidation With Vertical Drainage

PROCEDURE [redacted] DETERMINING RATE OF CONSOLIDATION FOR TWO LAYER SYSTEM

1. AVERAGE THE SOIL COEFFICIENTS OF TWO SOIL LAYERS:

   $K_{AVE} = \frac{R_H + 1}{R_H + R_K}(K_A)$ (PERMEABILITY)

   $m_{V_{AVE}} = \frac{R_H R m_v + 1}{R_H + 1}(m_{VB})$ (COEFFICIENT OF VOLUME COMPRESSIBILITY)

   $C_{V_{AVE}} = \frac{K_{AVE}}{m_{V_{AVE}}(\gamma_W)}$ (COEFFICIENT OF CONSOLIDATION)

2. COMPUTE EQUIVALENT DRAINAGE DISTANCE (H'):

   $H' = (H_A + R_K H_B)$ (SINGLE DRAINAGE)

   $H' = \frac{1}{2}(H_A + R_K H_B)$ (DOUBLE DRAINAGE)

3. COMPUTE EQUIVALENT THICKNESS RATIOS:

   $D_A = \frac{H_A}{H'}$, $D_B = \frac{R_K H_B}{H'}$

4. DETERMINE VALUES OF PERCENT CONSOLIDATION ($\bar{U}$) AT VARIOUS TIMES (t) FOR TOTAL THICKNESS ($2H$) USING NOMOGRAPH IN FIGURE 6-8

5. COMPUTE $\Delta\bar{U}_B$, PERCENT CONSOLIDATION OF PORTION $R_K H_B$ OF EQUIVALENT THICKNESS H', FROM EQUATIONS:

   (1) WHEN $\bar{U} \leq 33.3\%$ AND $D_B \leq \frac{3\bar{U}}{100}$

   $$\Delta\bar{U}_B = D_B - \frac{100}{3\bar{U}}(D_B)^2 + \frac{10{,}000}{27\bar{U}^2}(D_B)^3$$

   (2) WHEN $\bar{U} \leq 33.3\%$ AND $D_B \geq \frac{3\bar{U}}{100}$ $\quad \Delta\bar{U}_B = \bar{U}$

   (3) WHEN $\bar{U} > 33.3\%$ $\quad \Delta\bar{U}_B = D_B - \left(\frac{100\bar{U}}{200}\right)\left[3(D_B)^2 - (D_B)^3\right]$

6. COMPUTE RATE OF CONSOLIDATION FOR EACH LAYER INDIVIDUALLY AS FOLLOWS:

   $\bar{U}_A = \frac{\Delta U_A}{H_A/H'}$ $\quad \bar{U}_B = \frac{\Delta\bar{U}_B}{R_K H_B/H'}$

   FOR SINGLE DRAINAGE, LAYER B IS ADJACENT TO PREVIOUS BOUNDARY AND, $\Delta\bar{U}_A = \bar{U} - \Delta\bar{U}_B$

   FOR DOUBLE DRAINAGE, LAYER B IS SUCH THAT $H_A > R_K H_B$ AND $\Delta\bar{U}_A = 2\bar{U} - \Delta\bar{U}_B$

7. COMPUTE AVERAGE RATE OF CONSOLIDATION FOR ENTIRE LAYER: $\bar{U}_{AB} = \frac{H_A\bar{U}_A + H_B\bar{U}_B}{H_A + H_B} = \frac{R_H\bar{U}_A + \bar{U}_B}{R_H + 1}$

EXAMPLE OF COMPUTATION OF RATE OF CONSOLIDATION

1. $K_{AVE} = \frac{2.33 + 1}{2.33 + 0.0224}(5.38 \times 10^{-5}) = 7.62 \times 10^{-5}$ FT/DAY

   $m_{V_{AVE}} = \frac{(2.33)(0.667) + 1}{2.33 + 1}(3.22 \times 10^{-5}) = 2.47 \times 10^{-5}$ FT²/LB

   $C_{V_{AVE}} = \frac{7.62 \times 10^{-5}}{2.47 \times 10^{-5}(62.4)} = 0.0494$ FT²/DAY

2. $H' = \frac{1}{2}(14 + 0.0224(6)) = 7.07$ FT.

3. $H_A/H' = \frac{14}{7.07} = 1.98$ $\quad \frac{R_K H_B}{H'} = \frac{0.0224(6)}{7.07} = 0.02$

4. DETERMINE $\bar{U}$ FROM FIG. 6-8, FOR EXAMPLE: AT $t = 0.25$ YEARS, $\bar{U} = 23.8\%$.

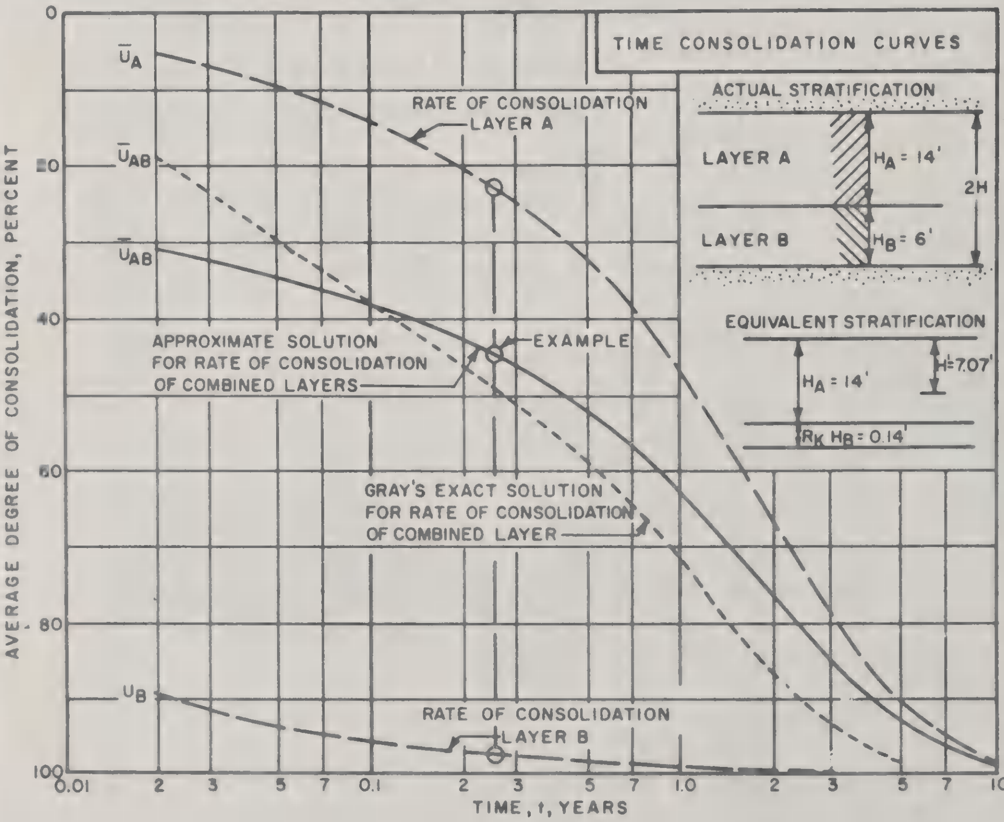
5. $\Delta\bar{U}_B = (0.02) - \frac{100}{71.4}(0.0004) + \frac{10{,}000}{15{,}294}(8 \times 10^{-6}) = 1.95\%$

   $\Delta\bar{U}_A = 2\bar{U} - \Delta\bar{U}_B = 47.6 - \Delta\bar{U}_B = 45.7\%$

6. $\bar{U}_A = \frac{47.5}{1.98} = 24.0\%$

   $\bar{U}_B = \frac{1.95}{0.02} = 97.5\%$

7. $\bar{U}_{AB} = \frac{2.33(24.0) + 97.5}{3.33} = 46.1\%$ AT $t = .25$ YEARS



KNOWN SOIL PROPERTIES

LAYER A:

$K_A = 5.38 \times 10^{-5}$ FT/DAY

$m_{VA} = 2.14 \times 10^{-5}$ FT²/LB

$C_{VA} = 0.04$ FT²/DAY

LAYER B:

$K_B = 2.41 \times 10^{-3}$ FT/DAY

$m_{VB} = 3.22 \times 10^{-5}$ FT²/LB

$C_{VB} = 1.20$ FT²/DAY

RATIOS:

$R_H = H_A/H_B = 2.33$

$R_K = K_A/K_B = 0.0224$

$R m_V = m_{VA}/m_{VB} = 0.667$

ASSUMING DOUBLE DRAINAGE

FIGURE 6-9
Time Rate of Consolidation for Two-Layer System

3. SECONDARY COMPRESSION.

a. Settlement Computation. Compute settlement from secondary compression following primary consolidation as follows:

$$\Delta H_{sec} = C_{\alpha}\,(H_t)\,\log \frac{t_{sec}}{t_p} \qquad (6\text{-}1)$$

where ΔH_{sec} = settlement from secondary compression.
C_{α} = coefficient of secondary compression.
H_t = initial thickness of compressible stratum.
t_{sec} = useful life of structure or time for which settlement is significant.
t_p = time to completion of primary consolidation.

See example in Figure 6-7. For preliminary estimate, use the values for the coefficient of secondary compression C_{α} given in Figure 3-5.

b. Combining Secondary and Primary Consolidation. If secondary compression is important, compute the settlement from primary consolidation separately, using an e-p curve that includes only compression from primary consolidation. A semilogarithmic straight line for the time-rate of secondary compression is made tangent to the final portion of the time curve for primary consolidation.

Section 6. METHODS OF REDUCING OR ACCELERATING SETTLEMENTS

1. GENERAL.
See Table 6-3 for methods of minimizing consolidation settlements. These include two general procedures: (a) removal or displacement of compressible material and (b) preconsolidation in advance of final construction.

2. REMOVAL OF COMPRESSIBLE SOIL.
Consider excavation or displacement of compressible materials for stabilization of fills that must be placed at the location of shallow soft strata.

a. Excavation. In harbors or waterways, dredging and hydraulic backfill may be economical for areas of wide extent. For excavation of soft materials on land, the expense and difficulty of ground water control must be considered.

b. Displacement. For highway fills partial excavation may be accompanied by displacement of soft foundation by weight of fill. Jetting in the fill or foundation and various blasting methods are used to facilitate displacement under fill load. Displacement is appropriate primarily for clean, coarse grained fill materials. For fills with high quality pavement, displacement methods are of uncertain value because of the probability of trapping pockets of soft material beneath the fill.

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 swell completed in the unloading stage.

b. Effect of Dewatering. If drawdown for dewatering extends well below subgrade, heave and consequent recompression are decreased by the application of capillary stresses. For excavation below the

TABLE 6-3
Methods of Reducing or Accelerating Settlements

Method	Applicability
Procedures for linear fills on swamps or compressible surface stratum:	
Excavation of soft materials.........	When compressible foundation soil extends to depth of about 10 to 15 ft, it may be practicable to remove it 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 materials.
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 toe 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 soils. 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 underlying soft foundation 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.
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 sand drains.	Used beneath highway fills to be paved and structural fills supporting light to moderate load buildings. Appropriate when a sufficient time is available during construction period for the consolidation process.
Surcharge fill with vertical sand 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.
Wellpoints placed in vertical sand drains.	Used to accelerate consolidation by providing a low boundary pressure in 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 load of fill in consolidating soft foundation. Applied in stabilization of cut banks in silts that cannot be stabilized by gravity drawdown. For settlement problems it is largely experimental.
Balancing load of structure by excavation.	Utilized in connection with mat or raft foundations that bottom in compressible material or where separate spread footings bottom in suitable bearing material overlying compressible stratum. Use of this method may eliminate deep, expensive foundations.

Note: For methods of stabilizing foundation by grouting and injection, see Chapter 15.

water table, the unit load removed equals the depth of excavation times total unit weight of materials if ground water level is restored after construction. If ground water pressures are permanently relieved beneath the foundation, the load removed equals the total weight of soil above the original water table plus the submerged weight of soil below original ground water.

4. **PRECONSOLIDATION BY SURCHARGE.** This procedure causes a portion of the total settlement to occur before commencing the structure. It is used primarily for fill beneath paved areas or structures with comparatively light column loads. For structures with heavy column loads, a thick fill of high rigidity may be required to avoid high stresses in compressible foundation soil.

a. **Elimination of Primary Consolidation.** Use the formula in the center panel of Figure 6-10 to determine surcharge load and percent consolidation under surcharge required to eliminate primary consolidation under final load. This computation assumes that consolidation time rate under the surcharge is equal to that under final load.

b. **Elimination of Secondary Compression.** Use the formula in the bottom panel of Figure 6-10 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 economy, surcharge load may threaten shear failure of the soft foundation. Analyze stability under surcharge by methods of Chapter 7, as follows:

(1) Where final thickness of a structural fill is small and individual column loads are heavy, surcharging generally is inappropriate.

(2) Column loads are limited by bearing capacity of the soft foundation. Heavy column loads may produce local settlements from stress concentrations beneath the footings.

5. **VERTICAL SAND DRAINS.** These consist of a column of pervious material placed in a cylindrical vertical hole in the compressible stratum, connected at the ground surface to a drainage blanket. Sand drains are utilized in connection with fills supporting pavements or low- to moderate-load structures.

a. **Characteristics.** Sand drains accelerate consolidation by facilitating drainage of pore water but do not change total compression of the stratum subjected to a specific load. Sand drains are laid out in rows, staggered, or alined to form patterns of equilateral triangles or squares. See Figure 6-11 for cross-section and design data for typical installations.

b. **Consolidation Rate.** Time rate of consolidation by radial drainage of pore water to sand drains is defined by time factor curves in lowest panel of Figure 6-6. For convience, use the nomograph of Figure 6-12 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 6-7.

c. **Sand Drain Design.** See Figure 6-13 for an example of design. For a trial selection of drain diameter and spacing, combine percent consolidation at specific time from vertical drainage with percent consolidation for radial drainage to sand drain. This combined percent consolidation $\overline{U}_c$ is plotted versus elapsed time for different drain spacing in the center panel of Figure 6-13. 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.** Commonly sand drain holes are driven with a closed-end mandrel. In this case soil in a surrounding annular space one-third to one-half the drain diameter in width is remolded and its stratification distorted by smear. Where sand drain design is based on consolidation tests of doubtful quality or an assumption of equal vertical and horizontal coefficient of consolidation, do not correct for smear or remolding. With detailed investigations of coefficient of consolidation in both

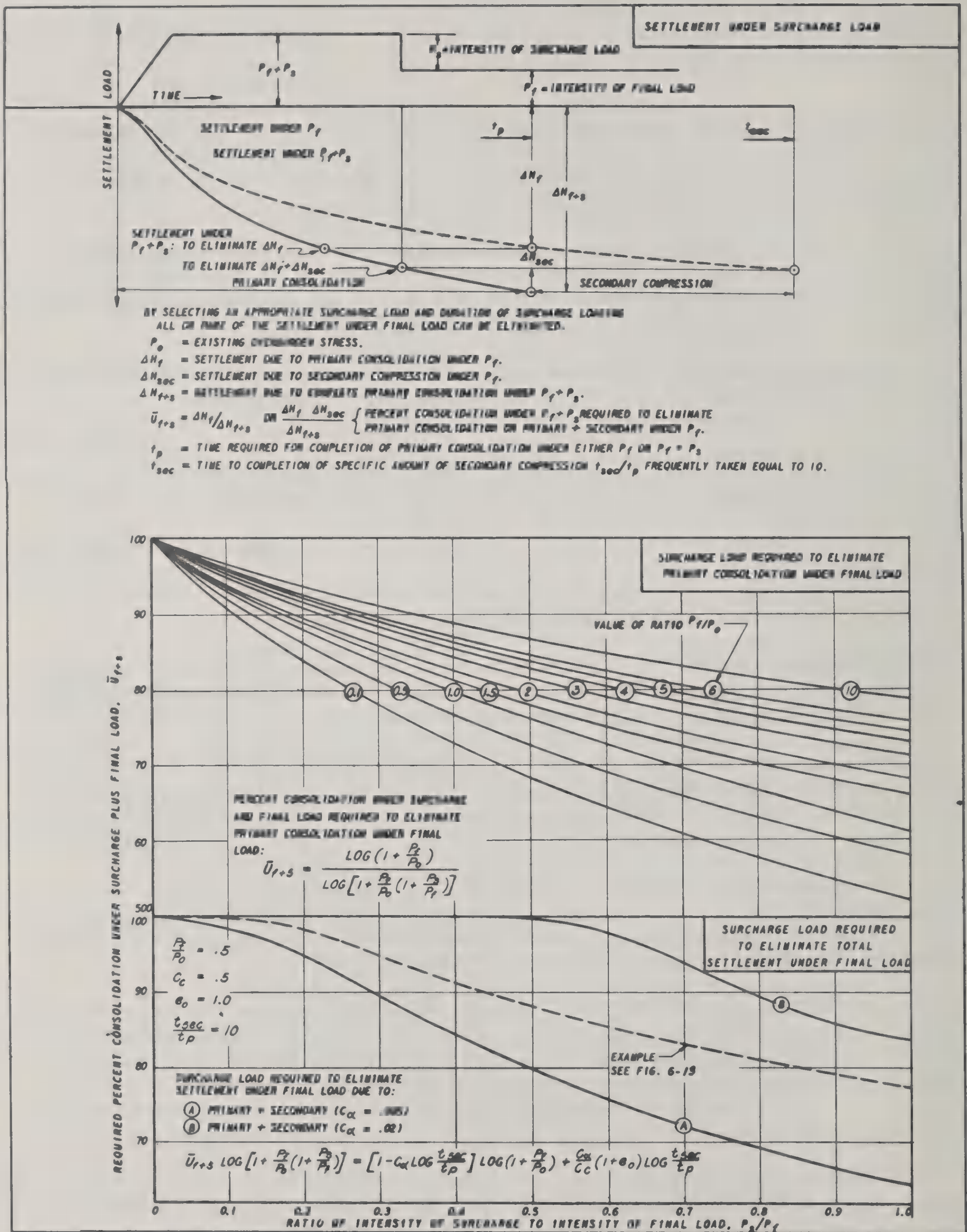
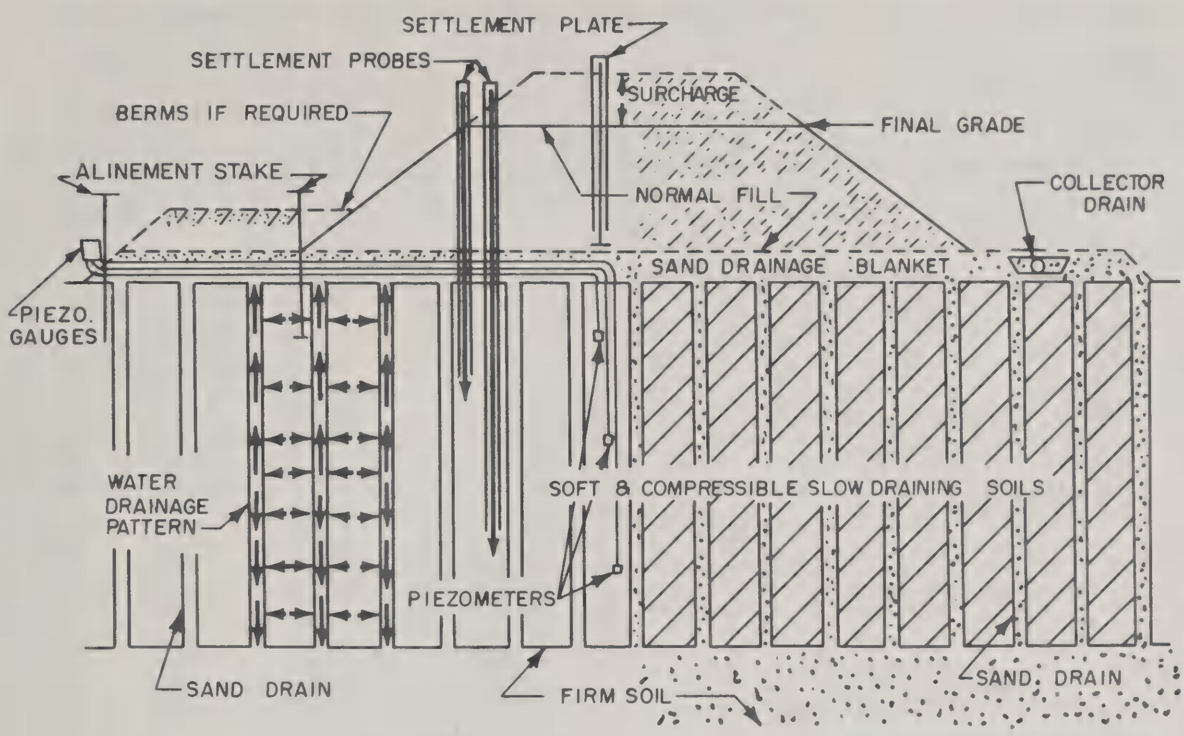


FIGURE 6-10
Surcharge Load Required to Eliminate Settlement Under Final Load



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;
Alinement stakes, T-shaped stakes placed at or outside embankment toe to observe lateral movement and heave.

FIGURE 6-11
Data for Typical Sand Drain Installations

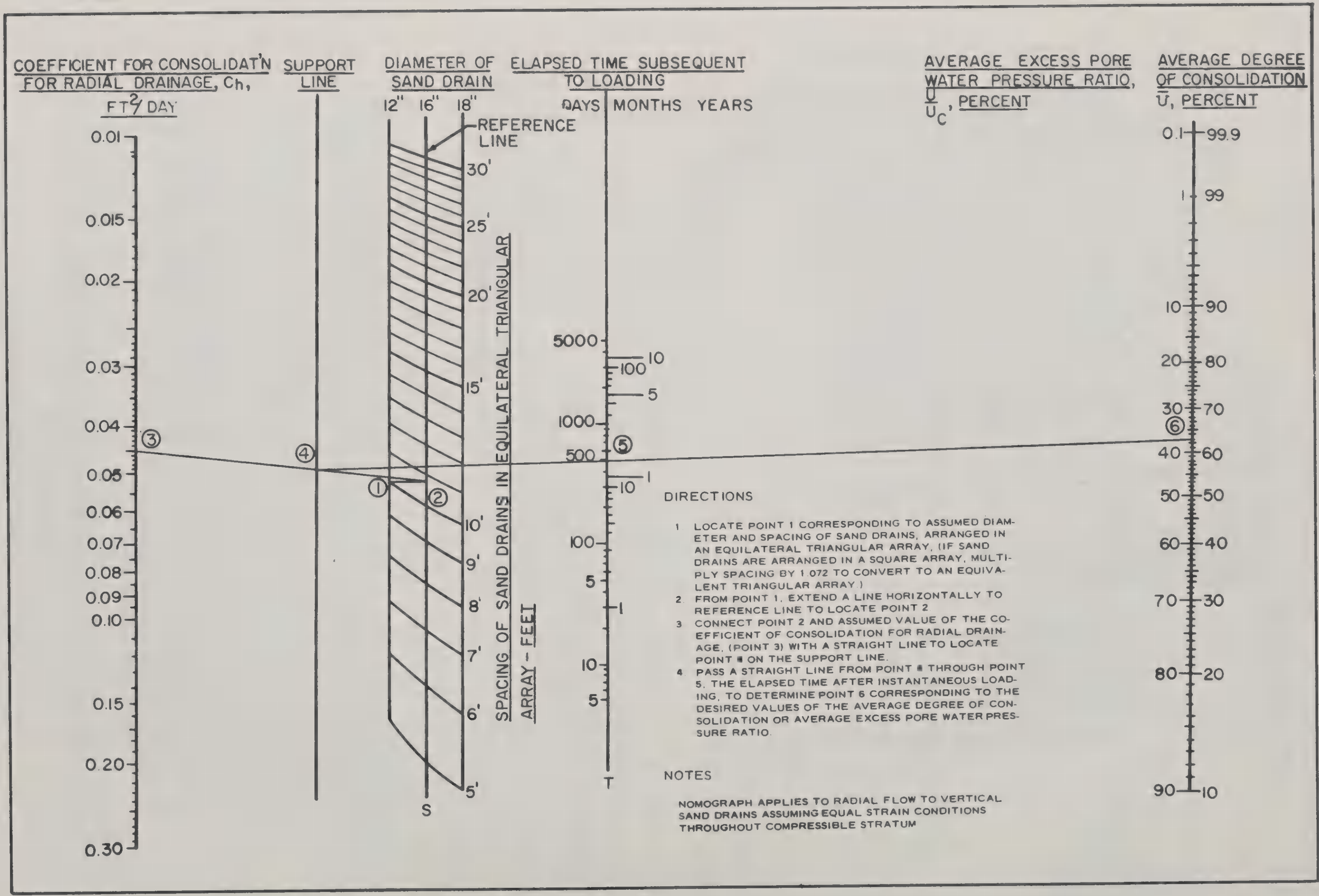


FIGURE 6-12
Nomograph for Consolidation with Radial Drainage to Vertical Sand Drain

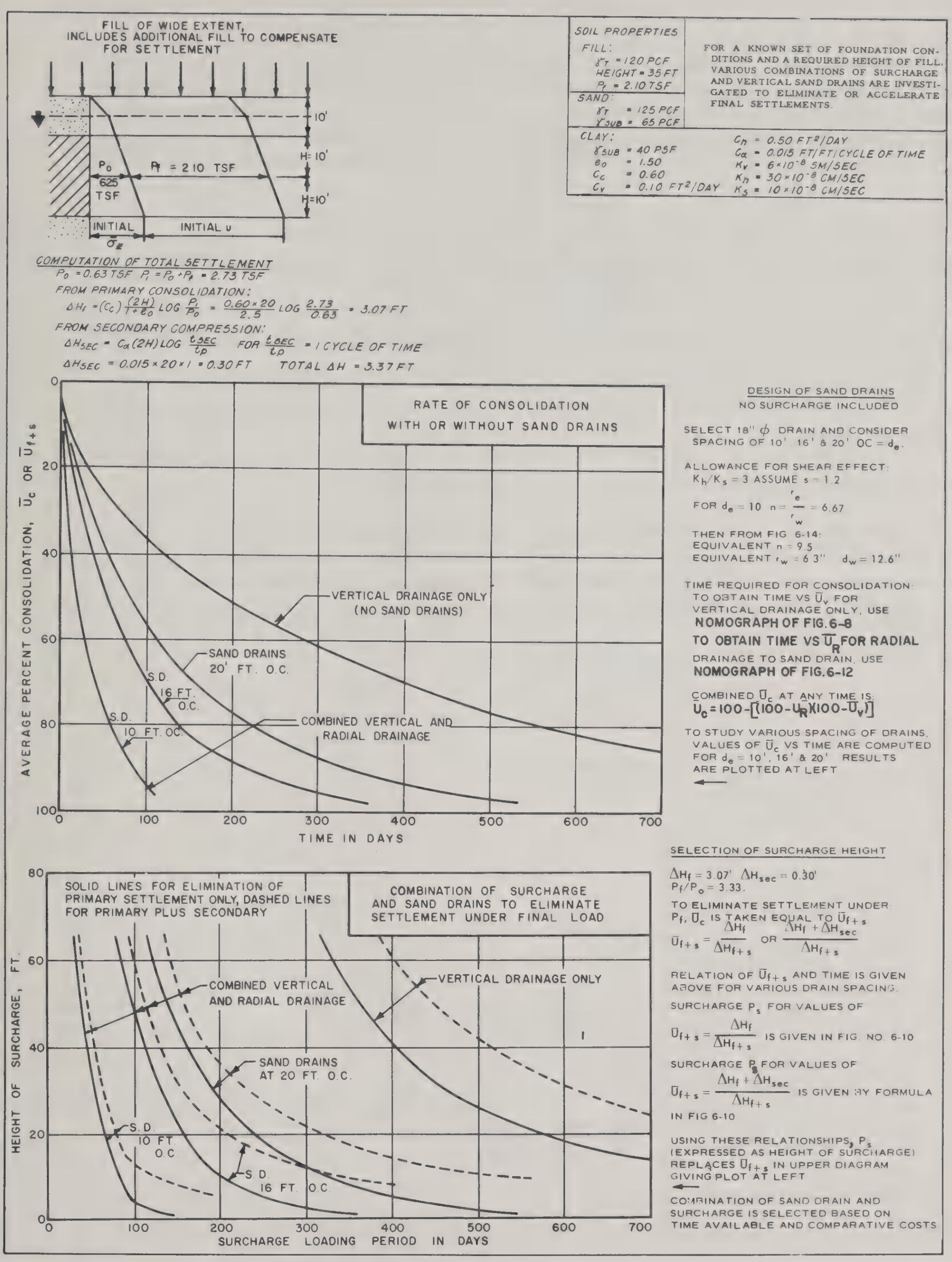


FIGURE 6-13
Example of Surcharge and Sand Drain Design

vertical and horizontal direction, include in the design a correction of smear in accordance with Figure 6-14.

e. Sand Drains Plus Surcharge. If construction time is limited, surcharge is placed above the final fill level in conjunction with sand drains to accelerate the required consolidation. Surcharge is frequently necessary when the compressible foundation contains material, such as peat, in which secondary compression predominates over primary consolidation. Percent consolidation under the surcharge fill required to eliminate a specific amount of settlement under final load is determined as shown in the lowest panel of Figure 6-13.

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 installations 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 8.

g. Construction Control Requirements. Control the rate-of-fill rise by observing pore pressure increase compared with pore pressure values allowable for stability. Check anticipated rate of consolidation by pore pressure and settlement measurements. If design rate is not achieved, consider the use of greater surcharge, closer spacing of drains in other sections, or increase of consolidation period before the structure is started.

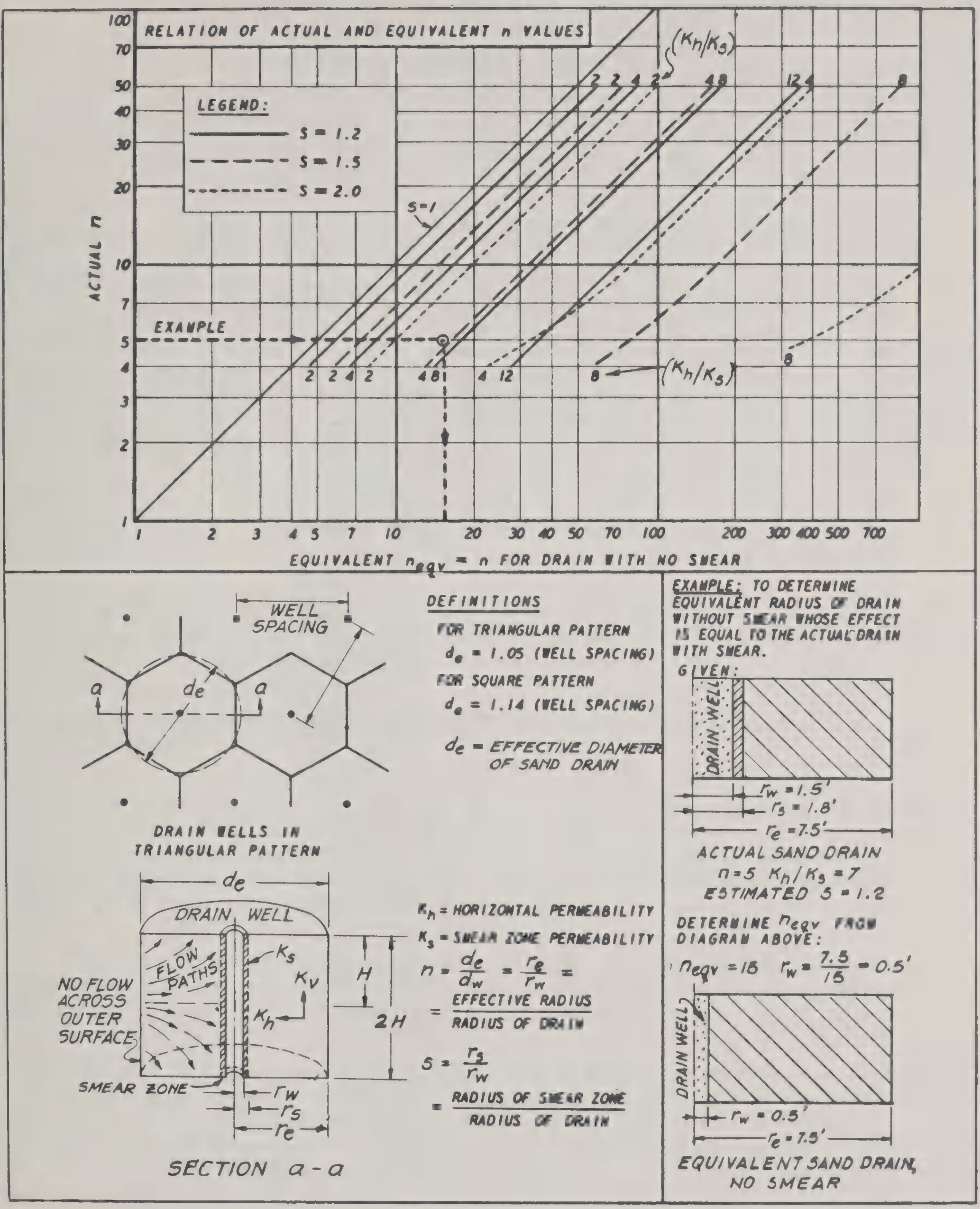


FIGURE 6-14
Allowance for Smear Effect in Sand Drain Design

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University of Illinois
B106 NCEL
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Urbana, Illinois 61801

CHAPTER 7. STABILITY ANALYSES

Section 1. INTRODUCTION

1. **SCOPE.** This chapter presents methods of analyzing stability of natural slopes and safety of embankments or rigid loads on soft foundations. Diagrams are included for approximate stability analysis, and procedures for slope stabilization are discussed.

2. **APPLICATIONS.** Overstressing a slope or foundation stratum may cause sudden failure with rapid displacement, or gradual shear strain, damaging structures or improvements. The possibility of movement is evaluated by comparing forces resisting failure to those causing failure. This ratio is the safety factor. Stability is also considered in determination of wall pressures where the triangular active wedge in the backfill is subject to failure.

Section 2. VARIETIES OF FAILURE

1. **FORM OF MOVEMENT.** Principal modes of failure are rotation on curved surface approximated by a circular arc, translation on a planar surface whose length is large compared to depth below ground, and displacement of wedge-shaped mass adjacent to yielding vertical surface.

2. **CAUSES OF MOVEMENT.** Failure follows changes in shear stress or shear strength that lead to unbalanced driving forces.

a. **Natural Slopes.** Unbalance of forces may be caused by a change in slope profile that adds driving weight at the top of a slide or decreases resisting force at its base, an increase of ground water pressure, resulting in a decrease of friction resistance in cohesionless soil or swell in cohesive material, or time-conditioned decrease in shear strength due to weathering, leaching, mineralogical changes, opening and softening of fissures, or continuing gradual shear strain.

b. **Foundations for Embankments or Structures.** Failure may be caused by an increase in applied load without a comparable increase in foundation shear strength or a decrease in foundation shear strength by rise of piezometric levels, opening of fissures, or continuing shear strain.

3. **EFFECT OF SOIL TYPE.** Figure 3-7 shows that the slope of the effective stress envelope for shear strength of clays of low plasticity equals the friction angle of loose or medium dense, coarse grained soils. Differences in stability of coarse and fine grained soils are caused primarily by the influence of pore pressure on strength. Shear strength cannot increase under load application unless effective stresses in the grain skeleton increase. This stress transfer occurs rapidly in pervious coarse grained soils but may be long delayed in impervious clays.

Section 3. ANALYSIS METHODS

1. **EFFECTIVE STRESS METHOD**. Utilize strength parameters c' and Φ' determined from effective stress test envelope or c and Φ from CU tests. Estimate pore pressures resulting from seepage and consolidation and apply these as boundary pressures normal to potential failure surface. Use effective stress analysis in the following situations:

(1) For long-term stability and drawdown in pervious, incompressible, coarse grained soils, use Φ', usually neglecting c'. Apply pore pressures from ground water or seepage only.

(2) For dense, moderately compressible soil, such as an earth dam embankment, use c' and Φ'. Apply only seepage or drawdown, or consolidation pore pressures if piezometers are installed to confirm pore pressures assumed in design.

(3) For compressible soils where some drainage occurs during load application, use c and Φ from CU tests. Apply ground water plus consolidation pore pressures, including an allowance for dissipation of hydrostatic excess pressures.

2. **TOTAL STRESS METHOD**. Use shear strength determined from undrained laboratory tests or from vane shear tests. Take Φ equal to zero. These strengths represent initial conditions without considering drainage of pore water during stress changes. Use total stress analysis for the following applications:

(1) Failures in slopes of normally consolidated or slightly preconsolidated clays, where little dissipation of hydrostatic excess pore pressures occurs prior to critical stability conditions.

(2) Analysis of embankment or structure load applied rapidly on a clay stratum where no provision is made to drain pore water.

3. **PROCEDURES**. See Table 7-1 for analysis methods and failure characteristics in natural slopes. See Table 7-2 for analysis methods and failure characteristics where time-conditioned changes in strength occur.

a. Rotational Failure, General Method. For details of slip circle analysis with movement on a surface approximated by a circular arc, use procedures described in Terzaghi and Peck, *Theoretical Soil Mechanics*.

b. Rotational Failure ($\Phi = 0$). For slopes in cohesive soils having approximately constant strength with depth, use Figure 7-1 to determine the safety factor. Utilize shear strength from U or UU tests, ignoring pore pressures, as follows:

(1) For slope in cohesive soils with strata of different strengths, determine centers of possible critical circles from Figure 7-2. Circles are tangent to interface between strata. Analyze these possible circles, applying the appropriate shear strength on sections of the arc in each stratum.

(2) With surcharge, tension cracks, or submergence of slope, apply corrections of Figure 7-3 to determine safety factor.

c. Rotational Failure (Φ and c strengths). For homogeneous material, use Figure 7-4 to compute safety factor with ground water below toe of slope. If ground water is near top of bank, compute approximate safety factor by using one-half the ordinary friction angle in the analysis, as follows:

(1) For materials where insignificant pore pressures are developed during shear, apply c' and Φ' from effective stress envelope.

(2) Where significant pore pressures are built up in shear, utilize c and Φ from CU tests.

(3) See Figure 7-3 for corrections for surcharge tension cracks, or submergence of slope.

d. Translation Failure. Where failure location is controlled by a relatively thin and weak layer, analyze stability of a translating mass with active and passive wedges by the method of Figure 7-5. See Figure 7-6 for an example of wedge analysis. To determine the overall safety factor of the entire mass,

TABLE 7-1
Analysis of Stability of Natural Slopes

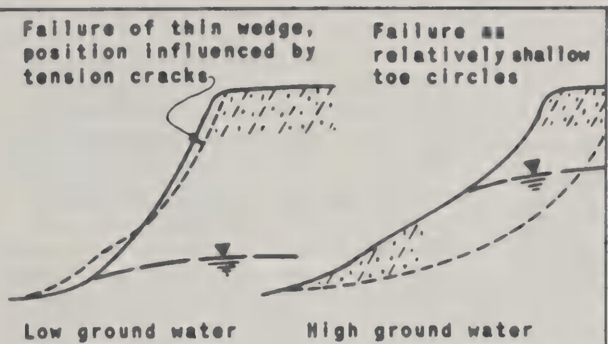
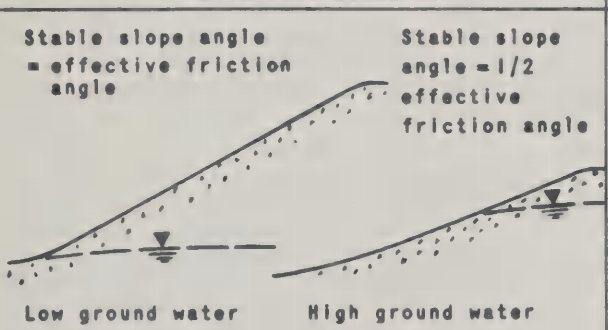
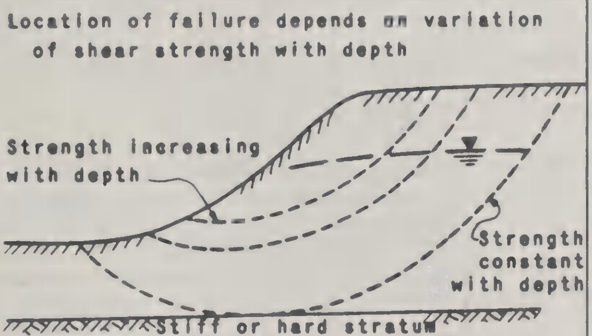
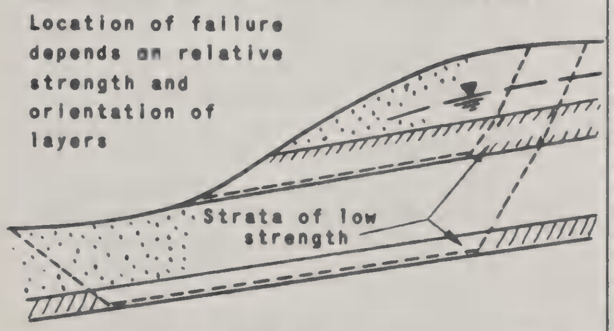
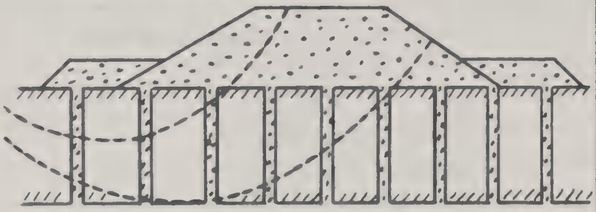
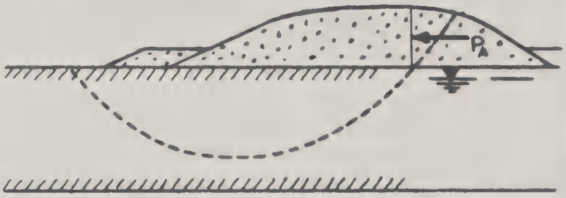
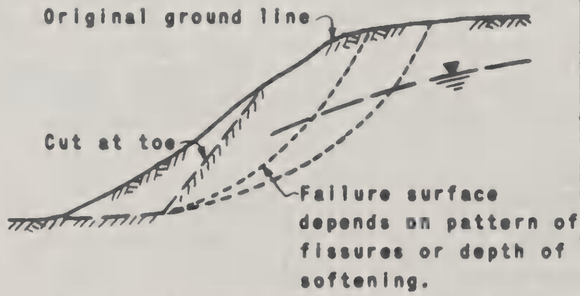
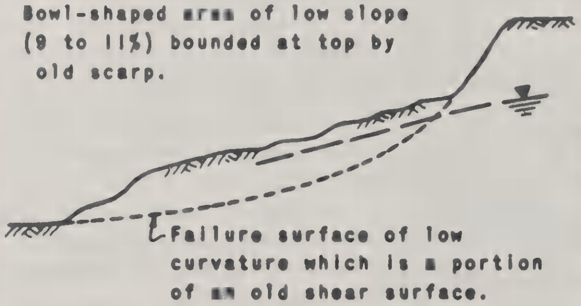
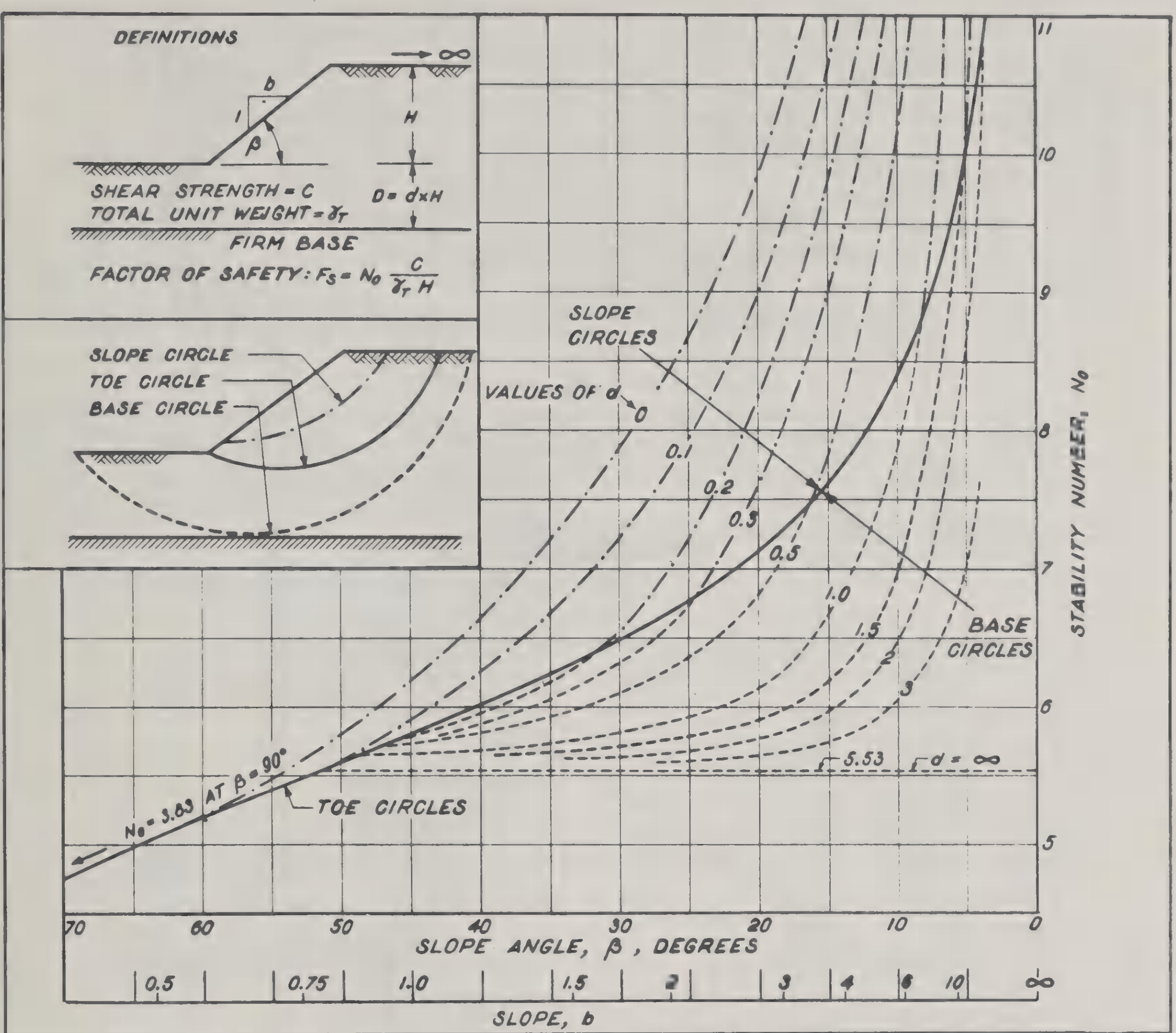
 (1) SLOPE IN COARSE GRAINED SOIL WITH SOME COHESION	With low ground water, failure occurs on shallow, straight, or slightly curved surface. Presence of a tension crack at the top of the slope influences failure location. With high ground water, failure occurs on the relatively shallow toe circle whose position is determined primarily by ground water elevation. Analyze with effective stresses using strengths C$'$ and ϕ' from effective stress envelope, applying ground water pressures and possible perched water levels from rainfall.
 (2) SLOPE IN COARSE GRAINED, COHESIONLESS SOIL	Stability depends primarily on ground water conditions. With low ground water failures occur as surface sloughing until slope angle flattens to friction angle. With high ground water, 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.
 (3) SLOPE IN NORMALLY CONSOLIDATED OR SLIGHTLY PRECONSOLIDATED CLAY	Failure occurs on circular arcs whose position is governed by theory, see Figure 7-2. Position of ground water 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 U or UU tests 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 from CU tests for fine grained strata and ϕ' from drained tests for cohesionless material.

TABLE 7-2
Analysis of Stability, Conditions Changing With Time

Location of failure depends on geometry and strength of cross section.  (5) FAILURE OF FILL ON SOFT COHESIVE FOUNDATION WITH SAND DRAINS	Usually, minimum stability obtains 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 Ø from CU test. Apply estimated hydrostatic excess pressures plus ground water pressures. For rapid construction without observation of pore pressures, use shear strength from U or UU test in total stress analysis.
Failure surface may be rotation on circular arc or translation with active and passive wedges.  (6) FAILURE OF STIFF COMPACTED FILL ON SOFT COHESIVE FOUNDATION	Usually, minimum stability obtains 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 Ø from CU tests with effective stress analysis, applying pore pressures of ground water only.
 (7) 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 Ø' from effective stress envelope. Analyze for long-term stability with C' = 0 and Ø' from effective stress envelope. With high water table, slope angle will approach 1/2 Ø' after a long period.
 (8) 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. If relatively limited movement has occurred, analyze with effective stresses utilizing Ø' from effective stress envelope with C' = 0. If large movements have occurred, strength will approach a lower limit of $Ø_r$. The true angle of internal friction, with C = 0.



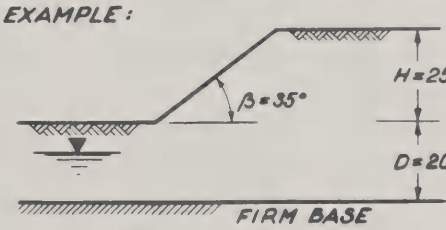
ASSUMED CONDITIONS:

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. 7-2.
2. FOR REDUCTION FACTORS FOR SUBMERGENCE, SURCHARGE OR TENSION CRACKS SEE FIG. 7-3.

EXAMPLE:



γ_T = 115 PCF C = 600 PSF $\phi = 0$

d = D/H = 20/25 = 0.80, β = 35°

$F_S = N_0 \frac{C}{\gamma_T H} = 5.8 \frac{(600)}{(115)(25)} = \underline{1.21}$

FAILURE IS BASE CIRCLE

FIGURE 7-1
Stability Analysis for Slopes in Cohesive Soils ($\phi = 0$)

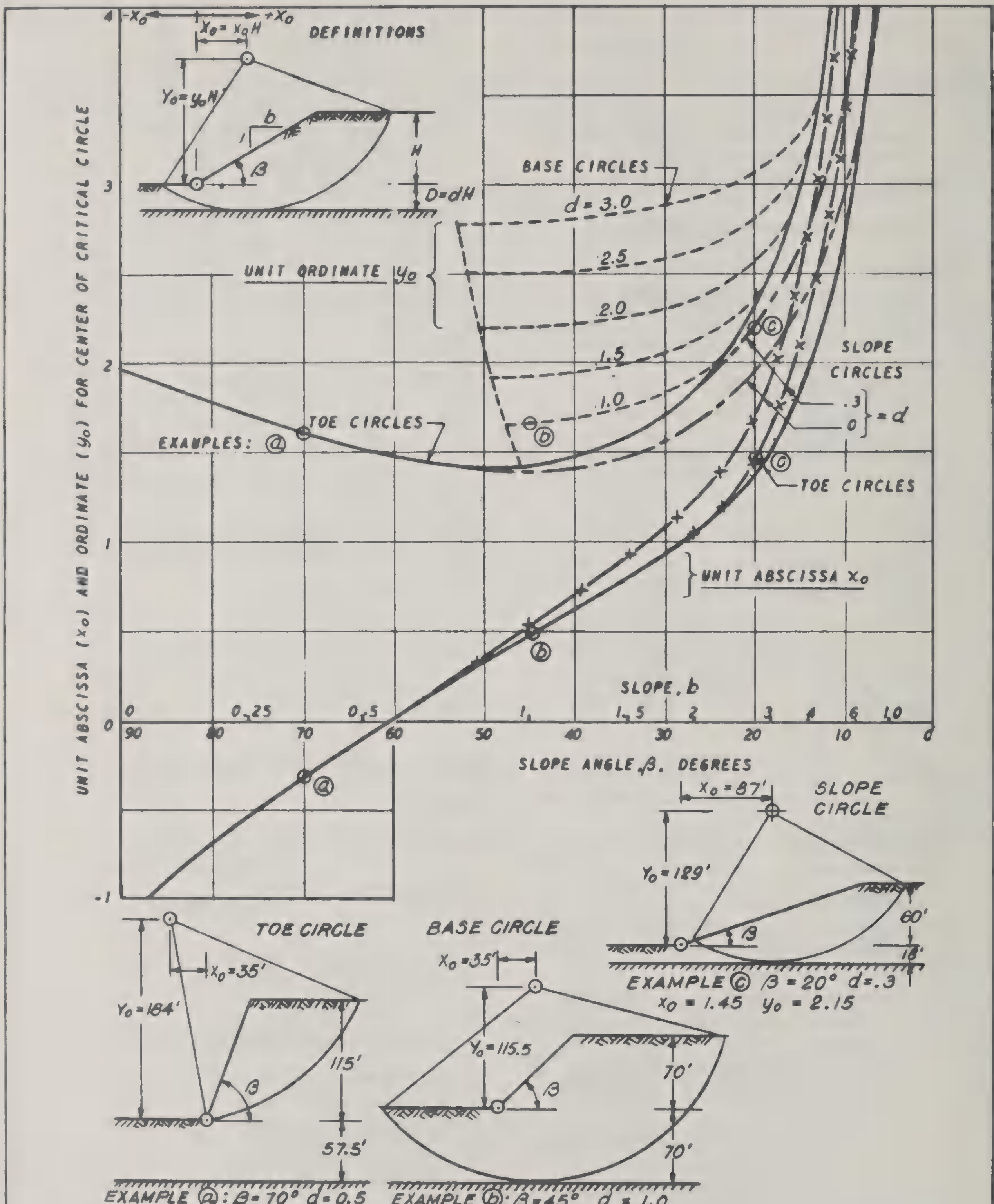
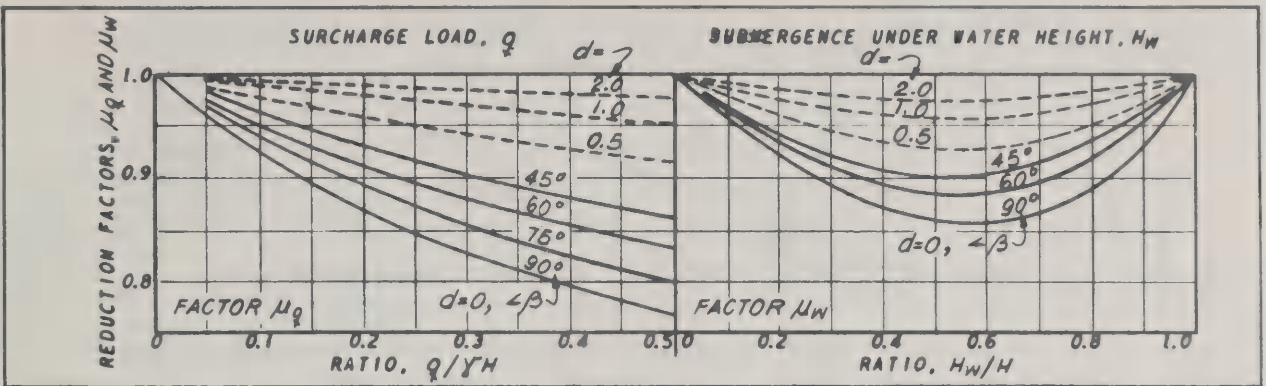
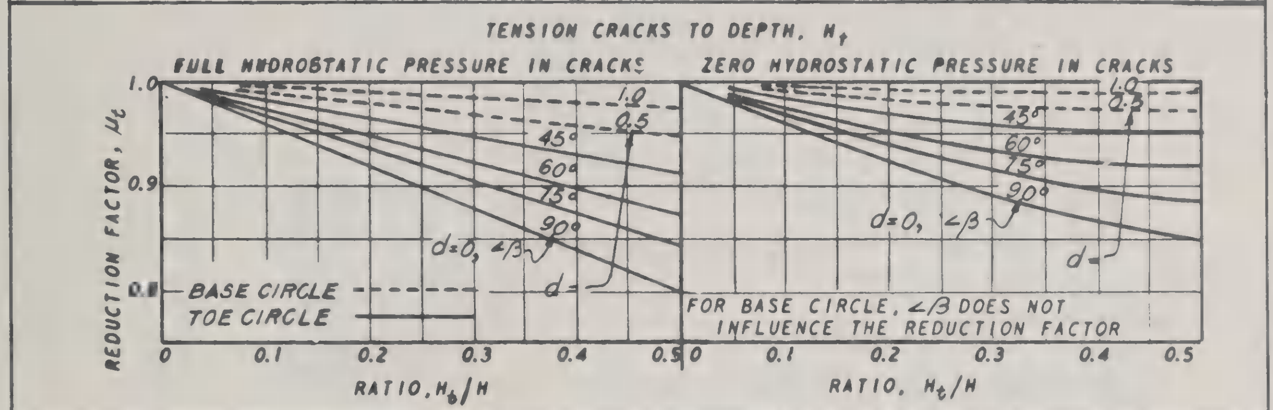
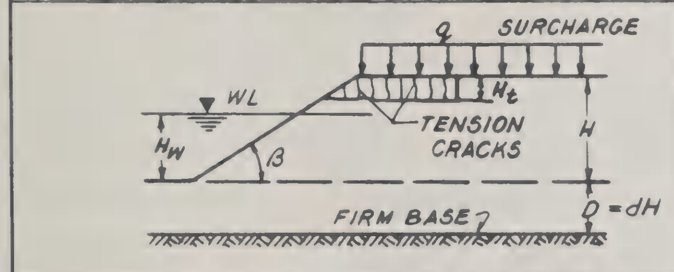


FIGURE 7-2
Center of Critical Circle, Slope in Cohesive Soil







INFLUENCE ON SAFETY FACTOR	
WHEN $\phi = 0$	$F_S = \dfrac{\mu_q \mu_W \mu_t N_o C}{\gamma H + q - \gamma_W H_W}$
WHEN $\phi > 0$	$F_S = \dfrac{N_{cf} C}{P_d}$ AND $\lambda_{c\phi} = \dfrac{P_e \tan\phi}{c}$ $P_d = \dfrac{\gamma_{sat} H + q - \gamma_W H_W}{\mu_q \mu_W \mu_t}$ $P_e = \dfrac{\gamma_{sat} H + q}{\mu_q}$

EXAMPLE WHEN $\phi = 0$

SEE "EXAMPLE," FIGURE 7-1

FOR: $q = 200$ PSF, $q/\gamma H = \frac{200}{115 \times 25} = 0.695$

$H_W = 15$ FT., $H_W/H = 15/25 = 0.60$

$H_t = 4$ FT., $H_t/H = 4/25 = 0.16$

THEN: $\mu_q = 0.940$

$\mu_W = 0.945$

$\mu_t = 0.985$ (ZERO HYDROSTATIC PRESSURE)

AND: $F_S = \frac{0.940 \times 0.945 \times 0.985 \times 5.8 \times 600}{115 \times 25 + 200 - 62.4 \times 15} = 1.45$

EXAMPLE WHEN $\phi > 0$

SEE "EXAMPLE," FIGURE 7-4

FOR: $q = 50$ PSF, $q/\gamma H = \frac{50}{120 \times 23} = 0.018$

$H_W = 5$ FT., $H_W/H = 5/23 = 0.217$

$H_t = 8$ FT., $H_t/H = 8/23 = 0.348$

THEN: $\mu_q = 0.995$, $\mu_W = 0.975$ AND $\mu_t = 0.990$ (FULL HYDROSTATIC PRESSURE)

AND: $P_d = \frac{120 \times 23 + 50 - 62.4 \times 5}{0.995 \times 0.975 \times 0.990} = 2600$

$P_e = \frac{120 \times 23 + 50}{0.995} = 2820$

$\lambda_{c\phi} = \frac{2820 \times .306}{280} = 3.08$ $F_S = \frac{14.1 \times 280}{2600} = 1.5$

FIGURE 7-3
Influence of Surcharge, Submergence, and Tension Cracks on Stability

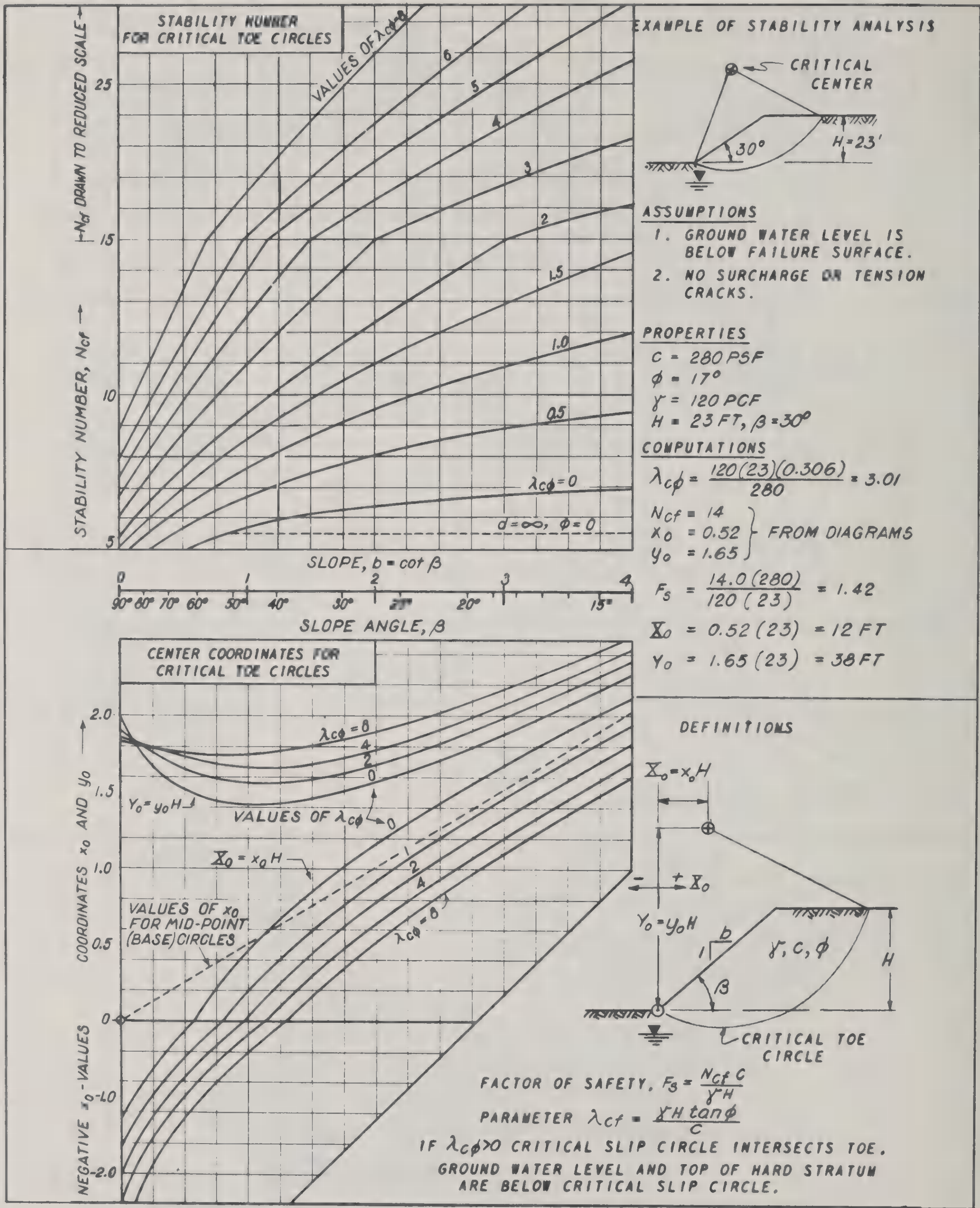


FIGURE 7-4
Stability Analysis for Slopes With ϕ and c.

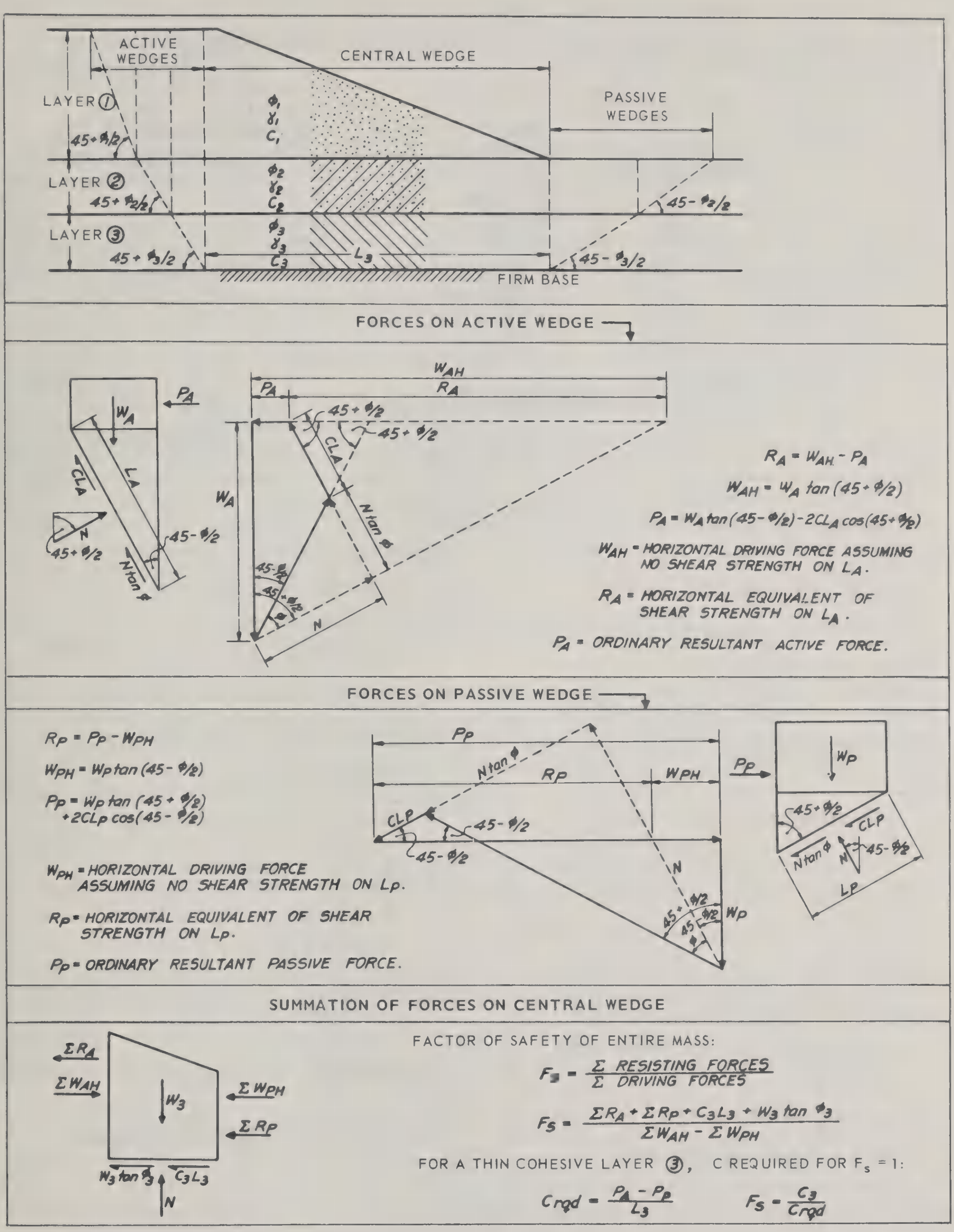
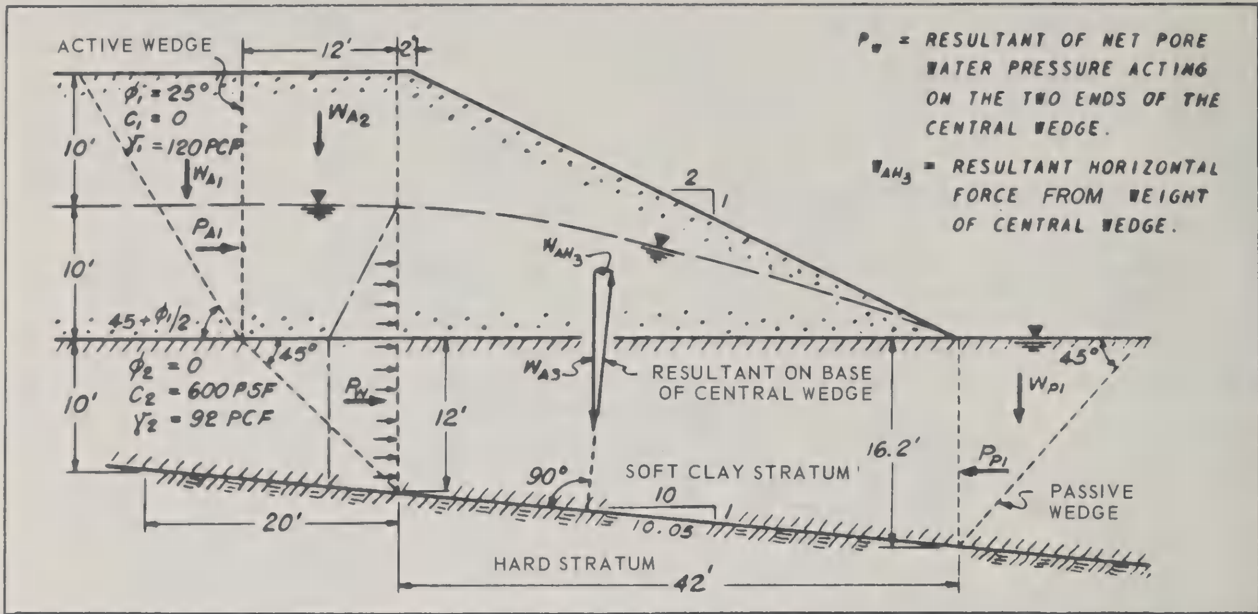


FIGURE 7-5
Analysis of Translational Failure



FORCES FROM ACTIVE WEDGE

W_{A1} = EFFECTIVE WEIGHT OF SOIL IN WEDGE

$$= 10 \times \frac{20+10}{2} \times \tan 32.5 \times 0.12 + \frac{1}{2} \times 10 \times 10 \times \tan 32.5 \times 0.058 = 13.3 \text{ K}$$

$$W_{AH_1} = W_{A_1} \tan(45 + \phi_1/2) = 13.3 \times \tan 57.5 = 20.88 \text{ KIP}$$

$$P_{A_1} = W_{A_1} \times \tan(45 - \phi_1/2) - 2CL_A \cos(45 + \phi_1/2) = 13.3 \times \tan 32.5 - 0 = 8.47 \text{ KIP}$$

$$R_{A_1} = W_{AH_1} - P_{A_1} = 20.88 - 8.47 = 12.41 \text{ KIP}$$

$$W_{A_2} = 12 \times 10 \times .12 + 12 \times 10 \times 0.058 + 12 \times \frac{12}{2} \times .030 = 23.51 \text{ KIP.}$$

$$W_{AH_2} = 23.51 \times \tan 45° = 23.51 \text{ KIP.}$$

$$P_{A_2} = 23.51 \times \tan 45° - 2 \times .6 \times 12 \times 1.414 \times \cos 45° = 9.11$$

$$R_{A_2} = 23.51 - 9.11 = 14.40 \text{ KIP.}$$

$$P_W = \frac{1}{2} \times 10 \times 0.062 \times 10 + 10 \times 0.062 \times 12 = 10.54 \text{ KIP.}$$

FORCES FROM PASSIVE WEDGE

W_{P_1} = EFFECTIVE WEIGHT OF SOIL IN WEDGE

$$= \frac{1}{2} \times 16.2 \times 16.2 \times 0.032 = 4.20 \text{ KIP.}$$

$$W_{PH_1} = W_{P_1} \tan(45 - \phi_2/2) = 4.20 \times \tan 45 = 4.20 \text{ KIP.}$$

$$P_{P_1} = W_{P_1} \tan(45 + \phi_2/2) + 2CL_p \cos(45 - \phi_2/2) = 4.20 \times \tan 45 + 2 \times .6 \times \frac{16.20}{\cos 45} \times \cos 45 = 23.63 \text{ KIP.}$$

$$R_P = P_{P_1} - W_{PH_1} = 23.63 - 4.20 = 19.4 \text{ KIP.}$$

$$C_2 L_2 = .6 \times 42.0 \times \frac{10.05}{10} = 25.2 \text{ KIP.}$$

FORCES FROM CENTRAL WEDGE

$$W_{A_3} = \frac{20}{2} \times 42 \times .12 + \frac{12 + 16.2}{2} \times 42 \times 0.092 = 105.0 \text{ KIP.}$$

$$W_{AH_3} = 105.0 \times \frac{1}{10} = 10.5 \text{ KIP.}$$

SAFETY FACTOR

$$F_s = \frac{\Sigma R_A + \Sigma R_P + C_2 L_2 + W_{A_3} \tan \phi_2}{\Sigma W_{AH} + P_W - W_{PH}}$$

$$F_s = \frac{12.62 + 14.4 + 19.4 + 25.0 + 0}{21.20 + 23.51 + 10.5 + 10.54 - 4.20}$$

$$F_s = \underline{1.08}$$

FIGURE 7-6
Example of Stability Analysis of Translational Failure

combine forces as shown in the vector diagrams. To determine the ratio of available strength in the critical stratum to strength required for stability, summarize the resultant active and passive forces as shown in the bottom panel of Figure 7-5.

e. Embankments on Soft Clay. See Figure 7-7 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 7-7.

f. Structure Foundation on Clay. For approximate analysis of a structure foundation on thick stratum, use bearing capacity method of Figure 11-1. For approximate analysis with an upper clay layer of finite thickness, see Figure 11-5. For detailed study, check safety factor on trial surfaces with active and passive wedges and translating block.

g. Required Safety Factor. The following values should be provided for reasonable assurance of stability:

(1) Safety factor no less than 1.5 for permanent or sustained loading conditions.

(2) For foundations of structures, a safety factor exceeding 2.0 is desirable to limit movements necessary for strength mobilization or local plastic strains at foundation edge. See Chapter 11 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.

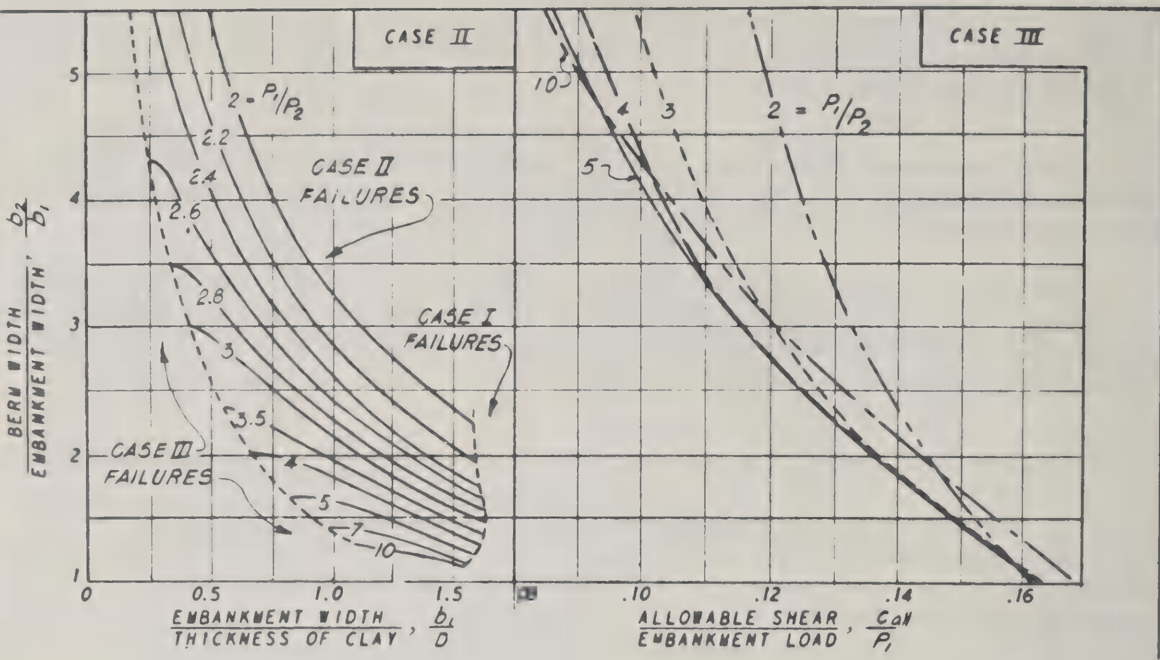
Section 4. PORE PRESSURE ANALYSIS

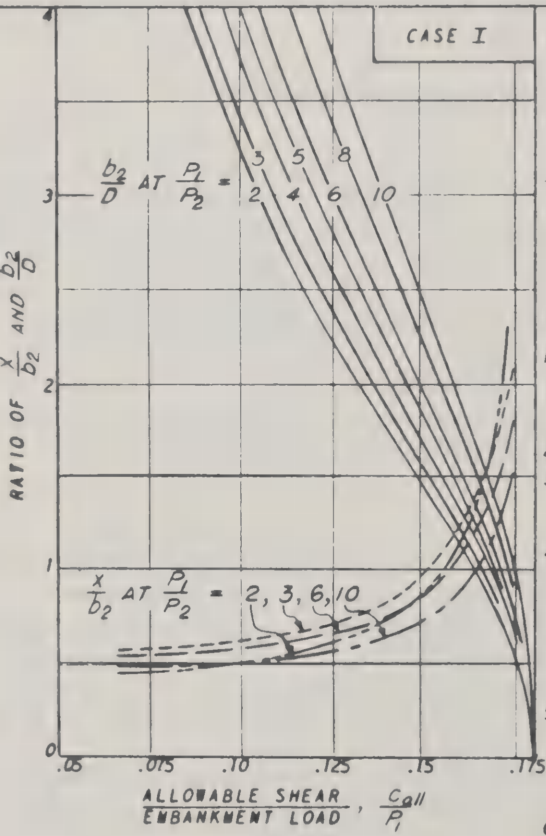
1. **PROCEDURES.** See Table 7-3 for pore water pressures that may be present in various situations before start of failure. In materials where no pore pressures are developed during shear, or where buildup of pore pressures is observed and controlled in the field, evaluate pore pressures as shown in Table 7-3 and apply them in effective stress analysis with C' and Φ' strengths. Where additional pore pressures are developed during shear of compressible impervious materials, utilize pore pressures of Table 7-3 in effective stress analysis with strengths C and Φ from CU tests.

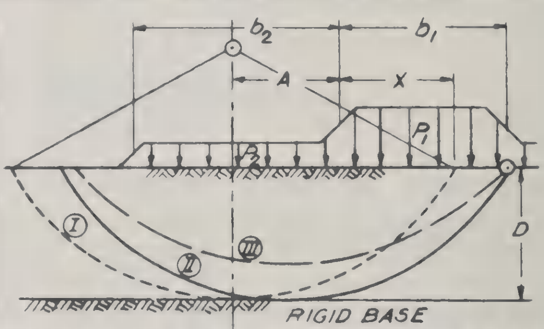
a. Seepage Pressures. Predict boundary pore pressures from flow net construction, or in case of rapid drawdown, by approximation of the pattern of equipotential lines. See Panels (3) and (4) of Table 7-3.

b. Construction Pore Pressures. In compressible fill materials placed at or above optimum moisture, construction pore pressures may develop during fill placement. Assume maximum pore pressures in the center of an impervious section of the embankment equal to the full theoretical value given by formula in Panel (2) of Table 7-3. Using judgment or examples of field observations, allow for dissipation of pore pressures on the periphery of the impervious section from drainage to pervious shells or foundation.

c. Consolidation Pore Pressures in Foundation. Where loading rate is rapid and no drainage relief is provided, consolidation pore pressures at the center of an impervious foundation may equal applied stresses at this level. Horizontal drainage in varved or lensed strata reduces pore pressures beneathe embankment centerline but may simultaneously increase pore pressures outside the toe. Where rate of construction is controlled or drainage is accelerated by vertical sand drains, estimate pore pressure dissipation by theory of consolidation. Apply reduced consolidation pore pressures plus ground water pressures in effective stress analysis using C and Φ from CU tests. When drainage allowance is included in design, provide piezometers for field observations to confirm pore pressure assumptions.





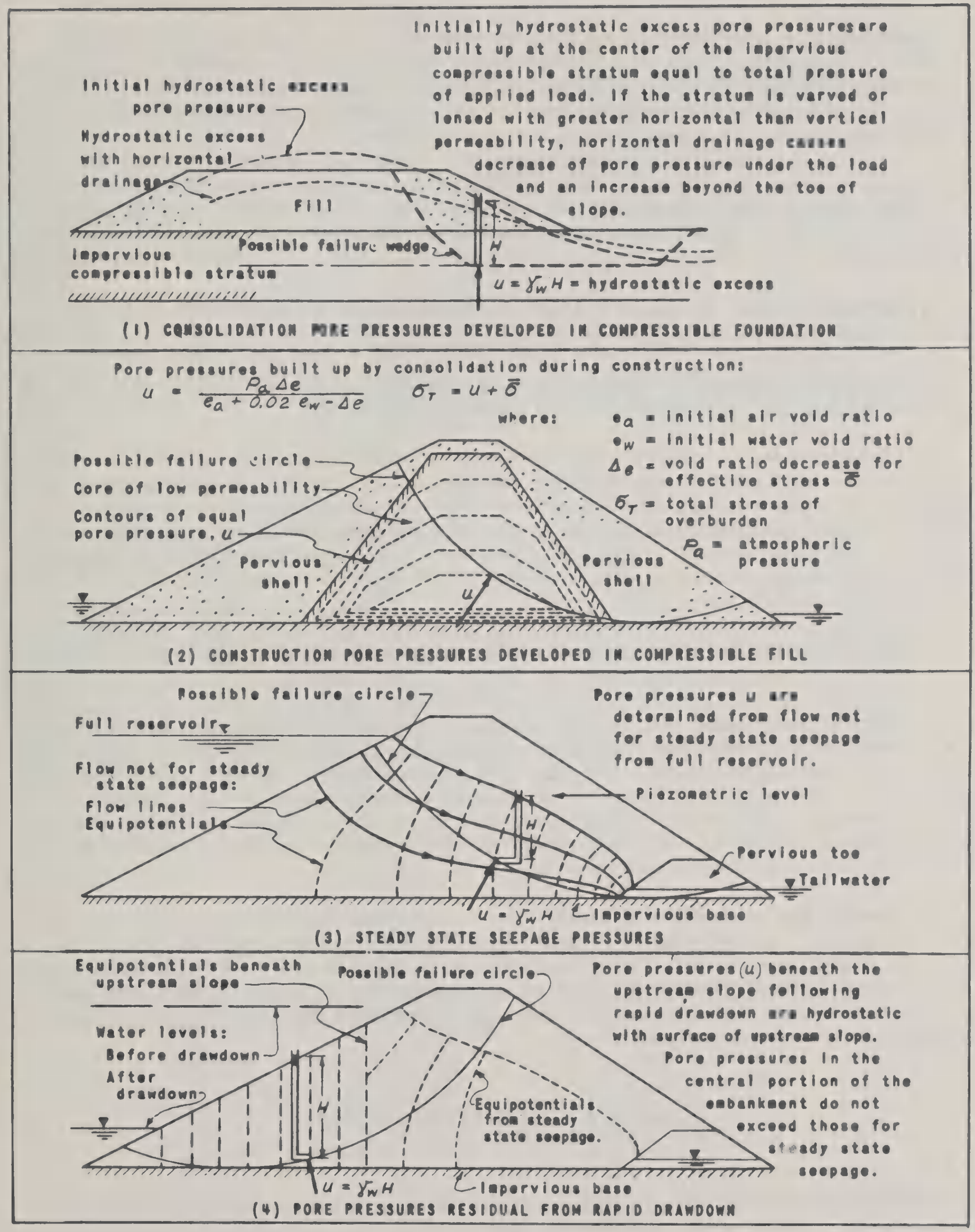


PROCEDURE FOR APPROXIMATE DESIGN OF BERMS

1. GIVEN b_1, P_1, D. SHEAR STRENGTH = C
   SAFETY FACTOR = F_s
2. ALLOWABLE SHEAR STRESS, $C_{all} = C/F_s$
3. DETERMINE P_2: $P_2 = P_1 - 5.5(C_{all})$
   P_2 MUST NOT EXCEED $5.5(C_{all})$
4. COMPUTE $\frac{b_1}{D}$ AND $\frac{P_1}{P_2}$ AND LOCATE $\frac{b_2}{b_1}$ IN UPPER LEFT GRAPH.
   IF CONDITIONS INDICATE CASE II FAILURE, $\frac{b_2}{b_1}$ IS DETERMINED FROM THIS GRAPH.
5. IF CONDITIONS INDICATE CASE I OR CASE III FAILURE ENTER APPROPRIATE GRAPH WITH $\frac{C_{all}}{P_1}$ AND $\frac{P_1}{P_2}$ AND DETERMINE $\frac{b_2}{b_1}$.
6. FOR CASE I CHECK BY DETERMINING $\frac{x}{b_2}$.
   $x < b_1$ FOR CASE I FAILURE. $A = \frac{P_2}{P_1} \times b_2$

FIGURE 7-7
Design of Berms for Embankments on Soft Clays

TABLE 7-3
Pore Pressure Conditions for Stability Analysis



(1) CONSOLIDATION PORE PRESSURES DEVELOPED IN COMPRESSIBLE FOUNDATION

(2) CONSTRUCTION PORE PRESSURES DEVELOPED IN COMPRESSIBLE FILL

(3) STEADY STATE SEEPAGE PRESSURES

(4) PORE PRESSURES RESIDUAL FROM RAPID DRAWDOWN

Section 5. PROBLEMS IN SPECIAL MATERIALS

1. SENSITIVE CLAYS.

a. Strength Problem. For clays with natural moisture content higher than the liquid limit, sensitivity and loss of strength on remolding are great. Small shear strains in such material may cause sudden decrease in strength and a flow slide, frequently of large dimensions.

b. Determination of Shear Strength. Undisturbed samples of the highest quality are required for tests. Consolidated-undrained tests (CU) are inappropriate because consolidation of even slightly disturbed material will result in significant decrease in moisture content and consequent overestimation of strength. Determine strengths by unconsolidated-undrained tests or by vane shear.

2. OVERCONSOLIDATED, FISSURED CLAYS. 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 in fissured clays, disregard cohesion c' determined from effective stress envelope and utilize Φ' only.

b. Old Slide Masses. Movements in old slide masses on relatively flat slopes frequently occur as gradual depth creep. Exploration may show the failure mass to be stiff or hard with high penetration resistance; but a narrow failure plane of low strength with slickensides or fractures may be undectected. In such locations avoid construction which involves regrading and ground water rise that may upset a delicate equilibrium already achieved. Depending on past magnitude of movement, effective shear strength ranges between the friction angle for effective strength Φ' and the true friction angle Φ_r with no cohesive resistance. See Figure 3-7.

3. LOOSE, COHESIONLESS SOILS. Special problems arise with two materials: (a) saturated, uniform fine sands, and (b) unsaturated loessial silts and fine sands of low density.

a. Saturated, Loose Sands. Uniform, silty fine sands with rounded grains, D_{10} size smaller than 0.1 mm, uniformity coefficient less than 5, and relative density less than about 40 to 50 percent, are sensitive to flow slides even at slopes as flat as 12° to 15°. Toe erosion, collapse of the loose grain structure by shock force, or small shear strain from drawdown, transfers overburden load to pore water pressure, practically eliminating shear resistance. No laboratory test methods reliably determine the condition for which sands are sensitive to liquefaction. Where loose, saturated fine sands are involved in waterfront construction, they should be stabilized by methods of Chapter 15.

b. Loessial Silts and Fine Sands. Usually where dry unit weights are less than about 85 pcf in situ, these materials are sensitive to loss of strength on saturation. In unsaturated condition, typical silty loess has these effective strengths: $c' = 700$ to 1,000 psf; $\tan \Phi' = 0.6$ to 0.65, and they stand on nearly vertical slopes for heights of 50 to 80 ft. Collapse on wetting practically eliminates cohesion and causes pore pressure increase. Evaluate saturation effect with unconsolidated-undrained tests, saturating samples under low chamber pressure prior to shear.

Section 6. SLOPE STABILIZATION

1. SLOPE PROFILE CONTROL. Regarding profile by flattening the slope, removing material producing driving forces, or adding material to increase resisting forces are common procedures. Evaluate influence of regrading by stability analysis of revised profile. Position stabilizing berms or fills to avoid a secondary stability problem underneath the berm and to prevent an increase in driving forces within the main slide mass.

2. **SEEPAGE CONTROL.** Seepage and ground water pressures are decreased by drainage structures. Surface control of drainage decreases infiltration to potential slide area. Drawdown increases effective stresses and eliminates softening of fine grained soils at fissures. Evaluate effects of drainage structures by flow net, applying altered boundary pore pressures to failure surface in stability analysis.

3. **STABILIZING STRUCTURES.** See Table 7-4 for methods of slope stabilization.

a. Application. Consider walls, caissons, and similar structures for slides with relatively small dimension in the direction of movement or for retaining steep toe slopes where failure will not extend backward into a larger mass.

b. Analysis. Retaining structures are misused where active forces on wall are caused by a failure wedge comprising only a small percentage of the total weight of the sliding mass. Such failures may pass beneath the wall entirely, completely unaffected by it, or 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 10-1), and possible failure on sliding surface at any level beneath the base of retaining structure.

4. **SPECIAL METHODS.**

a. Types. These include rock bolting, tieback systems, and procedures for increasing shear strength by injection or grouting.

b. Injection Methods. As with retaining structures, the increase in shear strength by injection must be considered as a portion of the overall stability problem. For relatively small masses to be stabilized, injection of a limited volume of soil has an important effect on total resisting forces. Where the total length of failure surface is many times larger than the portion of the length affected by injection, the influence of an increased shear strength will be proportionately small. To make injection useful, the ratio of improved strength to natural shear strength must be roughly equal to the ratio of total length of failure surface to the portion of the length strengthened by injection.

TABLE 7-4
Methods of Slope Stabilization

Grading of slope

Excavation at top of slope. Appropriate for deep masses of cohesive soil where rotational failure is likely. Less effective in creep movements or shallow transitional failures.

General flattening of slope. Applicable for deep masses of cohesive soil where rotational failure is likely and for relatively small slide areas where toe has been oversteepened.

Benching of slope. Appropriate on steep slopes where flattening is difficult and surface sloughing occurs. Assists in erosion control and in catching debris of small slides. Benches should be sloped to collect runoff and convey it off the slide.

Complete removal of unstable mass. Feasible for relatively small slide masses or shallow creep movements.

Earth fills at toe of slope. Placed as counterweight or stabilizing berm at locations where upward movement of slide mass may occur. Careful study is required so that addition of fill does not increase driving force.

Rock or gravel fills at toe of slope. Serves as counterweight, also controls toe erosion and seepage exit gradients, preventing backward erosion.

Surface drainage

Open ditch surrounding slide area. Useful in a variety of situations. Ditching within slide mass should be used with caution. Discharge should be directed away from potentially unstable areas.

Surface treatment by seeding, paving, or drainage blanket. Applied in critical locations for erosion control and to collect, control, or redirect surface seepage, preventing it from infiltrating unstable mass.

Regrading of surface to improve runoff. Useful for slide masses with sag ponds, depressions, or troughs that trap surface runoff. Frequently coupled with drainage ditches or trenches.

Sealing joint planes, cracks, and fissures. Appropriate for slides in stiff brittle materials where single large cracks or jointed zones can be sealed at the surface with mastic, clay, or cement slurry.

Subsurface drainage

Drainage by drilled horizontal or sloping drain holes. Frequently the most economical method of drainage. Appropriate for large and deep slide mass, particularly where pervious water bearing strata can be intercepted by drain holes before reaching failure zone. Thorough investigation of ground water conditions is a prerequisite.

Diversion of water by deep drainage trenches. Appropriate where shallow or perched ground water flow influences stability directly or delivers large quantity of water to slide mass. Usually limited to conditions where flow can be intercepted at depths of less than 10 to 15 ft.

Interception of seepage by tunnels. Useful for draining large slide masses and where structures of great value are threatened. Usually should be accompanied by drilled drainage holes extending outward from tunnel.

Vertical drain wells. Joined with horizontal drain holes or may be pumped directly to intercept strata or lenses of permeable material. Utilized in special cases to lower perched water table by draining into underlying pervious stratum. Accelerates consolidation and gain in strength of normally consolidated strata.

Retaining structures

Rock and earth fill buttresses at toe of slope. To be effective, volume of fill should be between 1/4 and 1/2 of volume of total slide mass and should extend between 5 to 10 ft below failure zone to provide adequate shear key.

Cribs or gravity retaining wall. Appropriate to prevent undercutting of toe or for resisting small slides, not suitable for large unstable masses.

Pile walls or caissons at toe of slope. Increases resistance on failure surface, must be driven to substantial depth below failure zone. Applicable only for slides of limited dimensions and frequently misapplied for slides of large extent. Thorough analysis should precede use of pile wall.

Barriers at toe of slope anchored by tieback to firm support. Useful in control of surface sloughing and small slides. Thorough analysis should precede use in large unstable masses.

Increase of Shear strength

Injection and cementation. Appropriate for soils of moderate permeability. Silicates, cement, or asphalt emulsion are utilized.

Freezing. Used as a temporary stabilization method during construction. Process is slow and costly.

Electro-osmosis. Applicable to soft fine grained materials. Costs may be high.

Compaction. Conventional procedure for embankment construction. Also utilized in removal and replacement of portion of unstable mass.

Miscellaneous

Rock bolting. Appropriate for steep slopes of weathered rock, fractured hard rock, or stiff soils where shallow sloughing or failure occurs on joint system.

Riprap protection. Prevents toe erosion and undercutting of slope at water's edge.

Blasting at toe of slope. Temporary expedient that interrupts slope surface and facilitates drainage of seeping water in lower portion of slide.

CHAPTER 8. SEEPAGE AND DRAINAGE ANALYSIS

Section 1. INTRODUCTION

1. **SCOPE.** This chapter covers analysis of flow quantity and ground water pressures associated with underseepage. Requirements are given for methods of drainage and pressure relief.

2. **RELATED CRITERIA.** Other criteria, relating to ground water utilization or control, but not covered in this chapter, will be found in the following sources:

Subject	*Source*
Pipes and conduits	NAVFAC DM-5
Subsurface drainage for airfields	NAVFAC DM-21
Subsurface drainage for highways	NAVFAC DM-5
Water wells and ground water supply	NAVFAC DM-5

Additional criteria for permanent pressure relief and seepage control beneath structures are given in Chapter 11.

3. **APPLICATIONS.** Seepage pressures are of primary importance in stability problems and in foundation design and construction. Frequently, drawdown of ground water is necessary for construction; in other situations pressure relief must be incorporated in permanent structures.

4. **INVESTIGATIONS REQUIRED.** For analysis of major underseepage problems determine permeability and piezometric levels by field observations; see Chapter 4 for techniques. Except for unusually homogeneous strata, laboratory permeability testing is of limited value in natural soils.

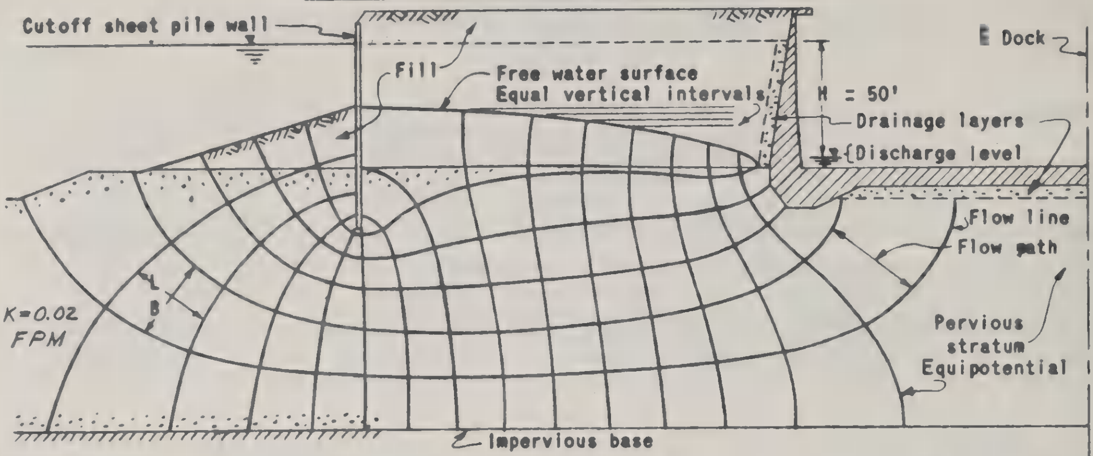
Section 2. SEEPAGE ANALYSIS

1. **FLOW NET.** See Figure 8-1 for method and example. 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, wellpoint, or shallow drainage installations placed in a rectangular layout whose length in plan is several times its width.

a. **Ground Water Pressures.** For steady state flow, water pressures depend on the ratio of mean permeability of separate strata and the anisotropy of layers, but are independent of absolute values of permeability. A carefully drawn flow net is necessary to determine piezometric levels within the flow field or position of the drawdown curve.

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. A rough flow net provides an accurate estimate of total flow. Uncertainties as to permeability values are a much more important limitation on accuracy.

Flow Net for Underseepage to Dry Dock



From flow net:

Number of flow paths = n_f = 6

Number of equipotential drops = n_d = 15

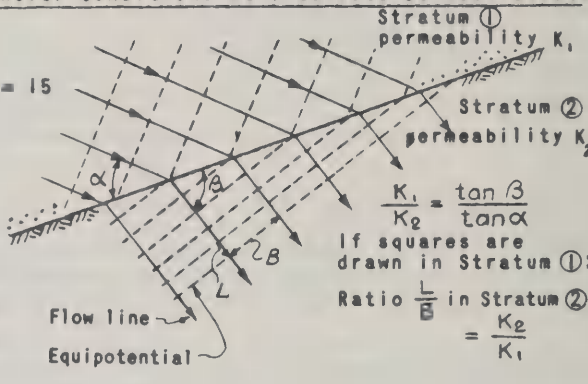
Underseepage per running foot of dock wall, $q = K\frac{n_f}{n_d}H$

$= 0.02\left(\frac{6}{15}\right)50 = 0.4$ CFM

For anisotropic soil:

Transform cross section by dividing dimensions in the direction of K_{max} by the factor $\sqrt{\frac{K_{max}}{K_{min}}}$

Transfer Conditions at Interface Between Strata



$$\frac{K_1}{K_2} = \frac{\tan\beta}{\tan\alpha}$$

If squares are drawn in Stratum ①:

Ratio $\frac{L}{B}$ in Stratum ② $= \frac{K_2}{K_1}$

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 than 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 8-1
Flow Net Analysis of Seepage

2. **THREE-DIMENSIONAL FLOW.** For analysis of flow quantity and drawdown to individual wells or to an array of wells, see Section 7; for additional information see Zanger, *Theory and Problems of Water Percolation* and, USCE, *Soil Mechanics Design, Seepage Control* (Bibliography).

Section 3. SEEPAGE CONTROL BY CUTOFF

1. **METHODS.** Procedures for seepage control include cutoff for decreasing seepage quantity or reducing exit gradients, and drainage or relief structures that increase flow quantity but reduce pore pressures or cause drawdown in critical areas. See Table 8-1 for methods of creating partial or complete cutoff.

2. **SHEET PILING.** A driven line of interlocking steel sheeting may be utilized for cutoff as a construction expedient or as a part of the completed structure.

a. Applicability. The following considerations govern the use of sheet piling:

(1) Sheeting is particularly suitable in coarse-grained material with maximum sizes less than about 6 in. 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. Its 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 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 g.p.m. per ft. of wall length for each 10 ft differential in head across sheeting, unless special measures are taken to seal interlocks.

b. Penetration Required. Seepage into an excavation beneath sheeting driven for partial cutoff may produce piping in dense sands or heave in loose sands. Piping occurs if the seepage exit gradient at subgrade equals about one. 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 ground water, sheeting must penetrate a sufficient depth below subgrade or supplementary drainage will be required at subgrade. See Figure 8-2 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 8-1 to obtain the equivalent cross section for isotropic conditions. See Figure 8-3 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) Drainage within an excavation is essential to prevent disturbance of subgrade if it consists of silty fine sand, silt, or varved-sand-silt-clay.

(4) 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 5.

TABLE 8-1
Cutoff Methods for Seepage Control

Method	Applicability	Characteristics and requirements
Sheet pile cutoff 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 sheet piling 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 locks 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 where the compacted cutoff may be imperfect or cracking of cutoff material is likely.
Grouted or injected cutoff.	Applicable where depth or character of foundation materials make sheet pile 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 Tables 15-2 and 15-3 for materials and methods.
Slurry trench method..	Suited for construction of impervious cutoff trench below ground water or for stabilizing trench excavation in connection with tremie placement of foundation walls.	Vertical sided trench is excavated by belowground water 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, well-graded backfill material is dropped through the slurry in the trench to form a dense mixture that is essentially incompressible after backfill is complete. Foundation wall is formed by concrete tremied to bottom of trench, displacing the slurry upward.
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 necessary. For data on mixed in-place piles, see Table 15-4.

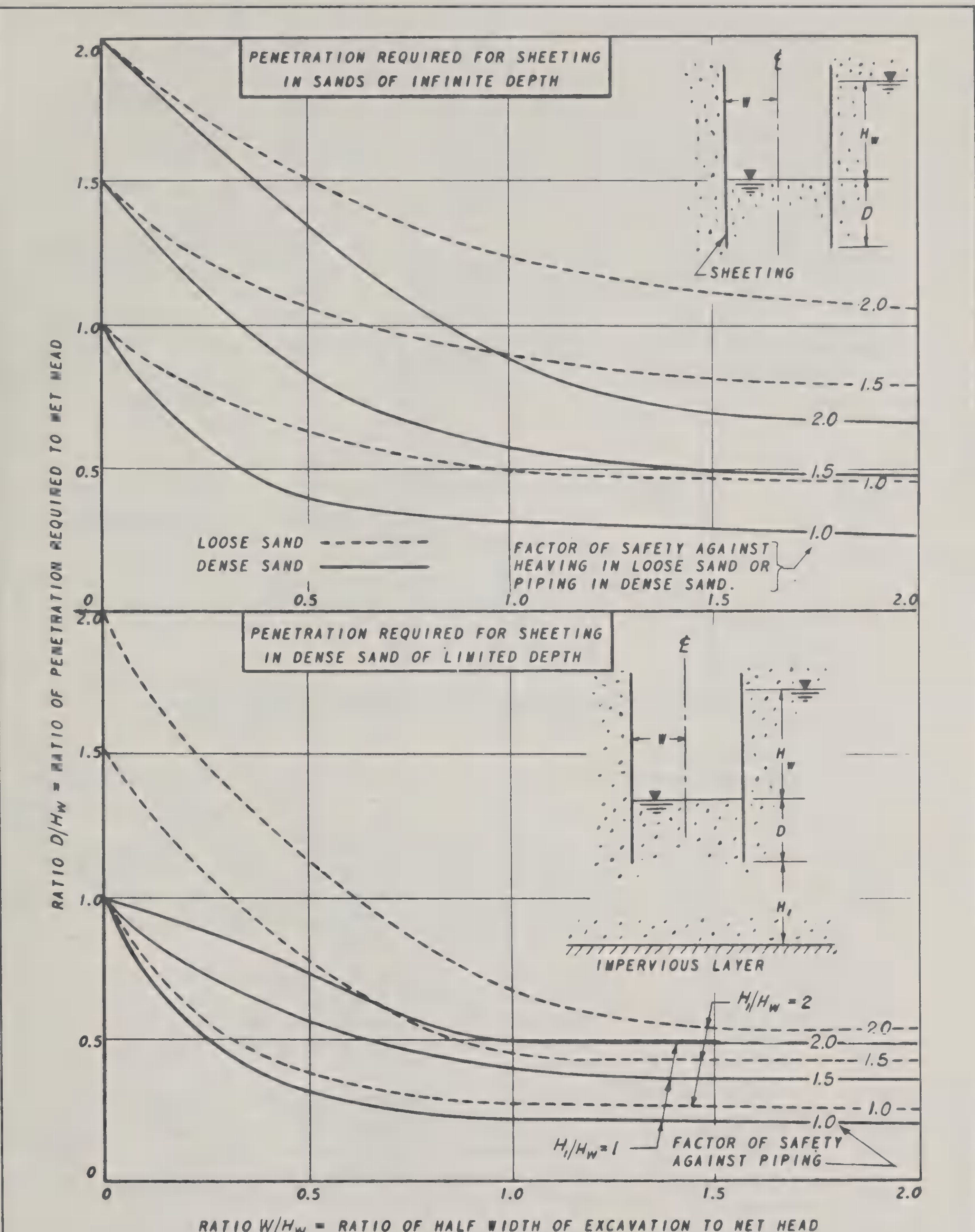
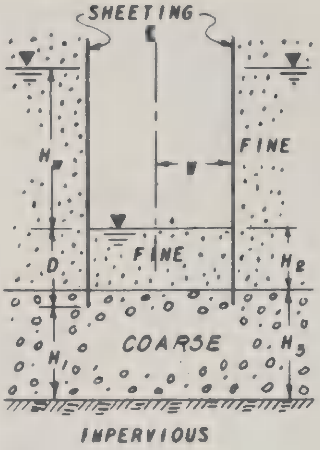


FIGURE 8-2
Penetration of Sheeting Required to Prevent Piping in Isotropic Sand

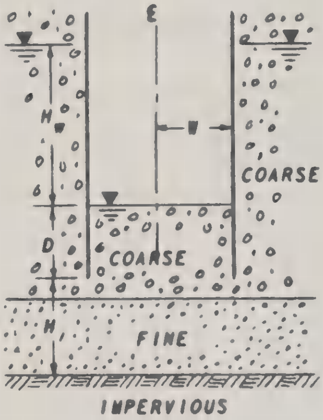


COARSE SAND UNDERLYING FINE SAND

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 FIG. 8-2.

IF TOP OF COARSE LAYER IS AT A DEPTH BELOW SHEETING TIPS GREATER THAN WIDTH OF EXCAVATION, SAFETY FACTORS OF FIG. 8-2 FOR INFINITE DEPTH APPLY.

IF TOP OF COARSE LAYER IS AT A DEPTH BELOW SHEETING TIPS 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).

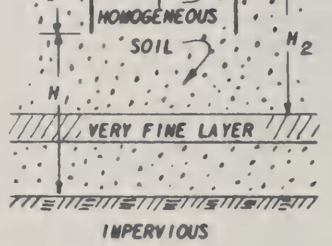


FINE SAND UNDERLYING COARSE SAND

PRESENCE OF FINE LAYER CONSTRICTS FLOW BENEATH SHEETING AND GENERALLY DECREASES SEEPAGE GRADIENTS IN THE COARSE LAYER.

IF TOP OF FINE LAYER LIES BELOW SHEETING TIPS, SAFETY FACTORS ARE INTERMEDIATE BETWEEN THOSE FOR AN IMPERMEABLE BOUNDARY AT TOP OR BOTTOM OF THE FINE LAYER IN FIG. 8-2.

IF TOP OF THE FINE LAYER LIES ABOVE SHEETING TIPS THE SAFETY FACTORS OF FIG. 8-2 ARE SOMEWHAT CONSERVATIVE FOR PENETRATION REQUIRED.



FINE LAYER IN HOMOGENEOUS SAND STRATUM

IF THE TOP OF FINE LAYER IS AT A DEPTH GREATER THAN WIDTH OF EXCAVATION BELOW SHEETING TIPS, SAFETY FACTORS OF FIG. 8-2 APPLY, ASSUMING IMPERVIOUS BASE AT TOP OF FINE LAYER.

IF TOP OF FINE LAYER IS AT A DEPTH LESS THAN WIDTH OF EXCAVATION BELOW SHEETING 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.

FIGURE 8-3
Penetration of Sheeting Required to Prevent Piping in Stratified Sand

(5) An outside open water source may be blanketed with fines or bentonite dumped through water or placed as a slurry. (See Table 8-2.)

(6) Evaluate the effectiveness of these measures by flow net analysis.

3. **GROUTED CUTOFF**. For grouting methods and materials see Chapter 15. Complete grouted cutoff frequently is 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 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. The impervious core of an embankment may be cracked by settlement or earthquake movements. Provide a chimney drain downstream of a core where core cracking is a possibility.

b. **Special Methods**. Impervious soil barriers formed in situ include:

(1) Overlapping mixed-in-place piles of cement and natural soil to form a cofferdam with some shear resistance surrounding an excavation.

(2) Concurrent excavation of a straight sided trench and backfilling with a slurry of clay mixed with natural soil. In certain cases tremie concrete may be placed, working upward from the base of a slurry-filled trench, to form a permanent peripheral wall.

Section 4. RESERVOIR IMPERMEABILIZATION

1. **METHODS**. Blanketing or lining of open water sources include methods to be used where wave action is insignificant, shown in Table 8-2, and protective linings that resist wave action, shown in Table 9-5.

2. **EARTH LININGS**. Where wave action is unimportant, compacted earth linings generally are suitable. Earth linings ordinarily are 2 to 4 ft thick on reservoir bottoms or sides below operating water levels. Permeability of thin earth blankets should be no greater than about 2×10^{-6} fpm. Permeability may be decreased by mixing a chemical dispersant with clayey materials for the blanket.

Section 5. SHALLOW DRAINAGE AND PRESSURE RELIEF

1. **METHODS**. Control of surface infiltration, drawdown of ground water to shallow depths, or relief or uplift pressures on slabs or walls are accomplished by drainage blankets or drainage trenches, or both in combination.

2. **SHALLOW DRAINAGE BLANKET**. See Figure 8-4 for typical installations.

a. **Permeability**. See Figure 8-5 for typical permeability coefficient 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 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 3-2.

Metz Reference Room
University of Illinois
B106 NCEL
208 N. Romine Street
Urbana, Ill...

TABLE 8-2
Impermeable Reservoir Linings

Method	Applicability and Procedures
Buried plastic liner.	Impervious liner formed of black colored polyvinyl chloride plastic film. Prepare surface for placement by raking to remove sharp stones. Where foundation is rough or rocky, place a layer 2 to 4 in. thick of fine grained soil beneath liner. Seal liner sections by bonding with plastic solvent with 6-in. overlap at joints. Protect liner by 6-in. cover of fine grained soil. On slopes add a 6-in. layer of gravel and cobbles 3/4 to 3 in. in size. Anchor liner in a trench at top of slope.
Bentonite seal.....	Bentonite is mixed with soil in a thin compacted lining, or placed beneath a protective soil cover, or placed under water to seal leaks that have been detected after reservoir filling. For placing dry, use 1 lb of bentonite for each square foot of lined area. For placing under water, bentonite may be poured as a powder or mixed as a slurry and flowed into the reservoir. Use at least 0.8 lb 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.
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 6-in. lift of clayey silt or clayey sand. Typical rate of application is 0.05 lb psf. Chemical and soil are mixed with mechanical mixer and compacted by sheepsfoot roller. Using a suitable dispersant, the thickness of compacted lining may be reduced to about 1 ft, the permeability of the compacted soil may be reduced to 1/10 of its original value.
Application of chemicals to increase adsorbed water capacity of clayey soils.	In calcium clays the water-holding capacity of the clay lattice may be increased by percolating salt water through it, resulting in expansion of the clay mineral lattice and decrease in permeability. Addition of chemicals such as resinous polymers to percolating water increases the adsorbed water on clay particles in soil surrounding the leak, causing swell and decreasing permeability.
Reservoir linings combining impermeability with slope protection.	Linings formed of materials that provide strength and rigidity to resist wave action and tractive forces. These include separated or articulated concrete slabs, monolithic concrete paving, pressure applied concrete lining, asphaltic paving, asphalt panels, steel plate, and thick compacted soil lining. See Table 9-5 for details of materials and methods. Selection of type depends on economy and severity of erosive forces. Frequently, a layer of drainage or filter material is required beneath these linings to control seepage outward through the embankment at breaks in the lining.

b. Drainage Capacity. Estimate the quantity of water which can be transmitted by a drainage blanket by Eq. 8-1:

$$q = K\ i\ A \qquad \text{(Eq. 8-1)}$$

where: q = quantity of flow
K = permeability coefficient
i = average gradient in flow direction
A = cross section area of blanket

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 drain pipe within the blanket.

(1) *Pressure Relief.* See bottom panel of Figure 8-6 for combinations of drain pipe spacing, drainage course thickness, and permeability required for control of flow upward from an underlying aquifer under a vertical gradient of 0.4.

(2) *Rate of Drainage.* See the top panel of Figure 8-6 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.

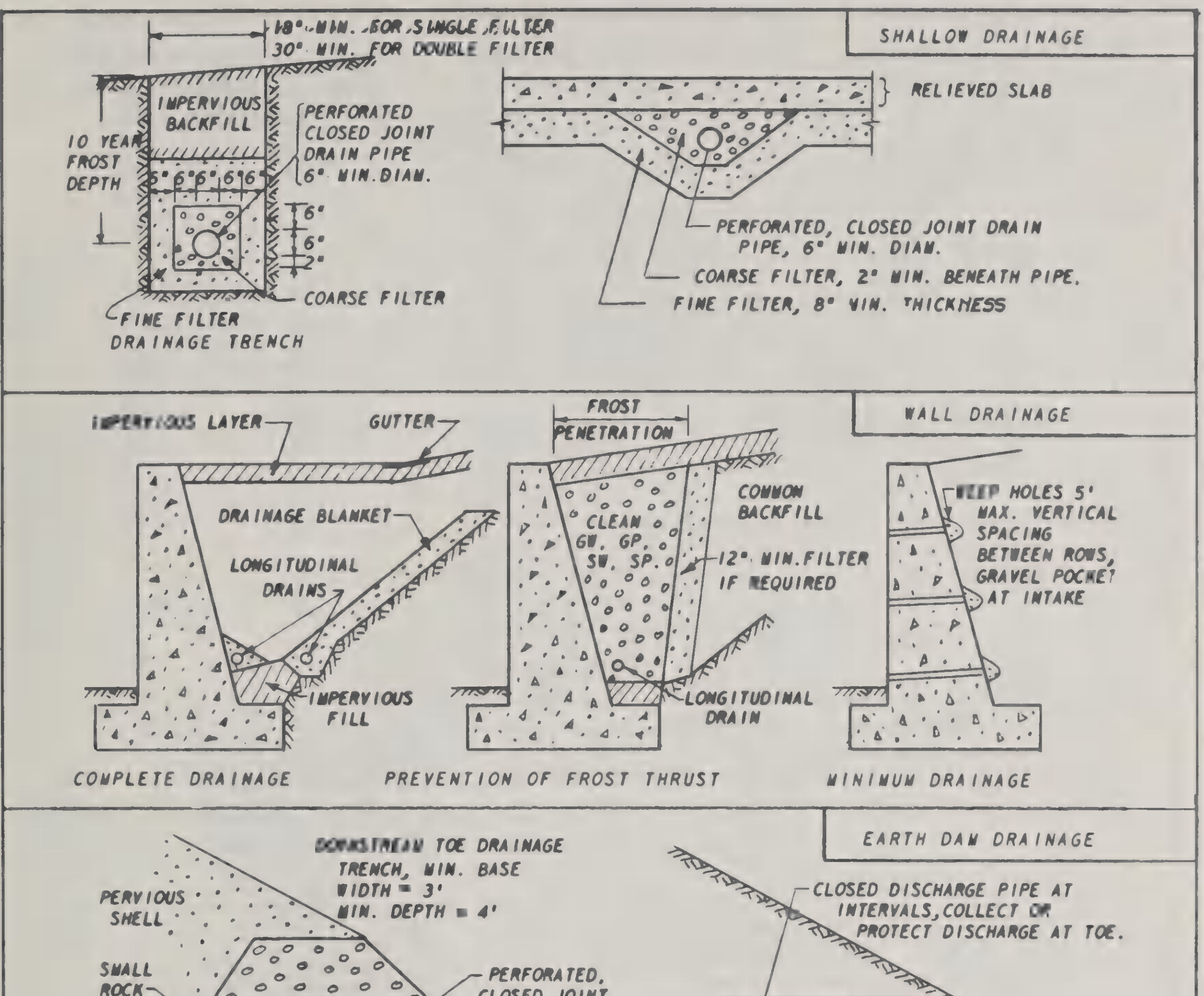


FIGURE 8-4
Filter Requirements for Drainage Structures

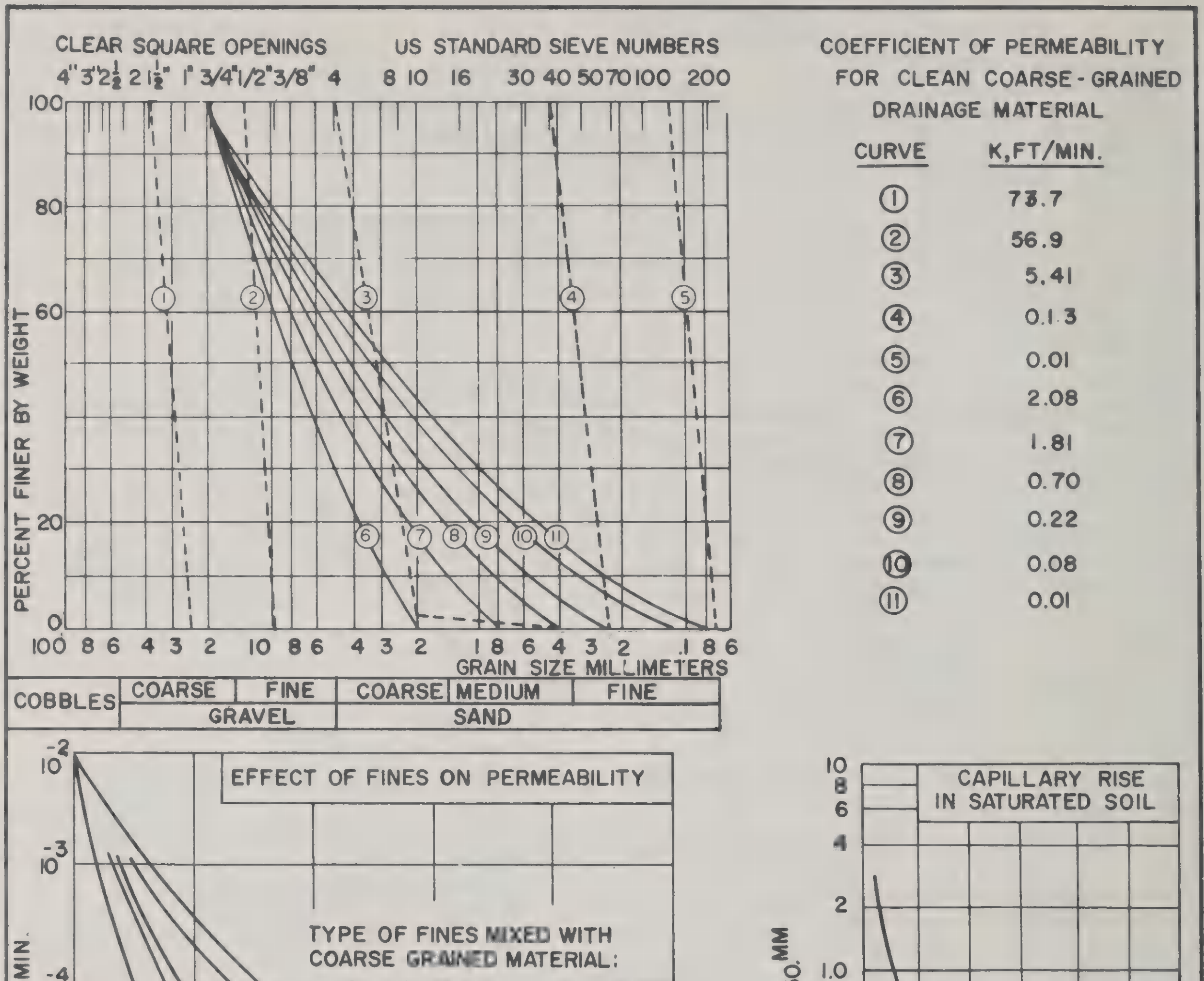
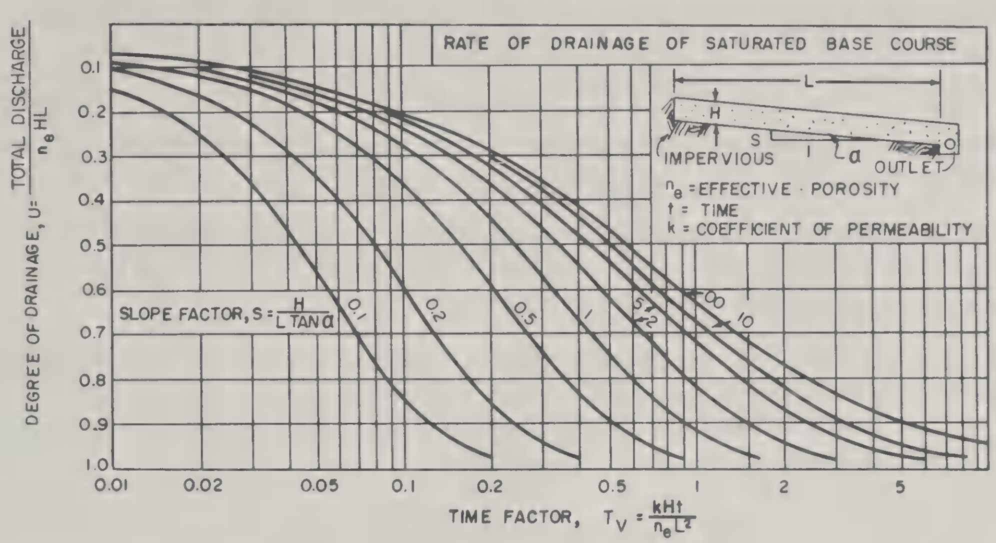


FIGURE 8-5
Permeability and Capillarity of Drainage Materials

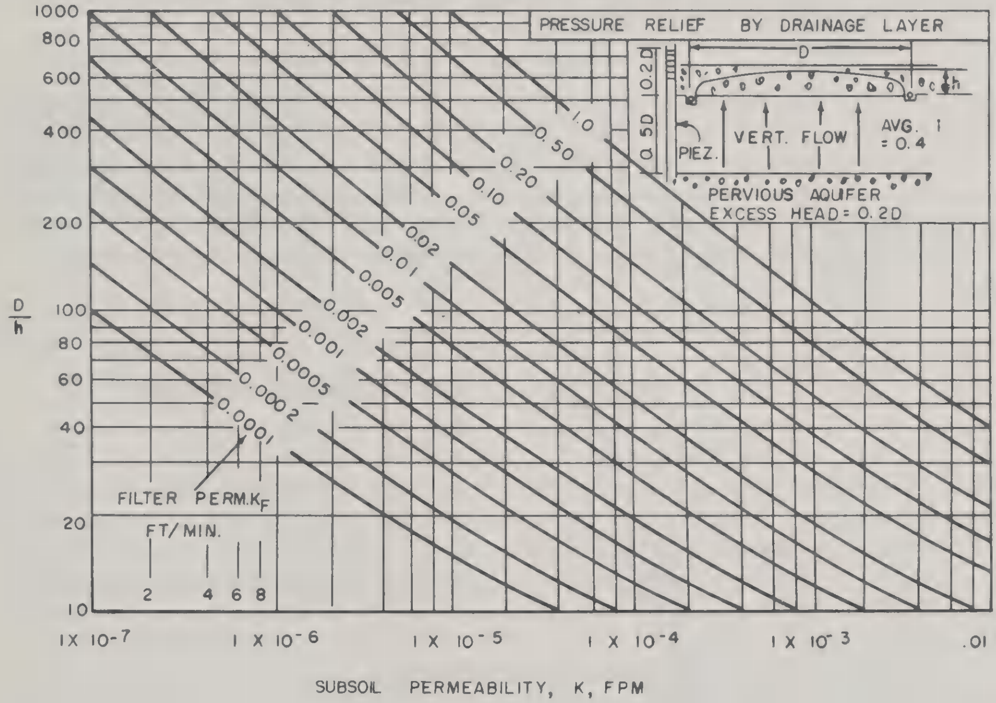


Select drain spacing, L, so that 50% drainage will be completed in 10 days.

$$t_{50} = \frac{T_v n_e L^2}{KH}$$

For estimate use: $t_{50} = \dfrac{n_e L^2}{2K(H + L \tan\alpha)}$

t_{50} drainage rate: $q = KH \dfrac{(H + L \tan\alpha)}{2L}$



Assumptions:
Steady seepage moves vertically upward from aquifer at depth 0.5D with average gradient = 0.4

To prevent breakout of seepage on ground surface, select filter permeability and thickness so that max. height of water in drainage layer (h) is less than filter thickness.

FIGURE 8-6
Analysis of Drainage Layer Performance

3. INTERCEPTING DRAINS. These drains consist of shallow trenches with collector pipes surrounded by drainage material, placed to intercept seepage moving horizontally in an upper pervious stratum, or to collect surface runoff and rainfall without ponding.

a. Control of Shallow Seepage. To design proper control drains, determine the drawdown and flow to drains by flow net analysis.

b. Control of Infiltration. See Figure 8-7 for requirements of intercepting drains placed to collect infiltration from rainfall without ponding. If sufficient capacity cannot be provided in trenches, add surface drainage facilities to prevent ponding.

4. PROTECTIVE FILTERS. See Figure 8-8 for protective filter design criteria. The use of filters in drainage structures is illustrated in Figure 8-4. Except for special cases, such as an underfloor drain beneath a floor and above a pressure slab, filter requirements apply to all permanent drainage structures in contact with soil, including wells.

a. Stability Requirements. The coarseness of a filter placed against a base is limited by the requirement for preventing intrusion of base fines into the filter. Concrete sand (ASTM C33) suffices as a filter against the majority of fine grained soils or silty and clayey sands. For nonplastic silt, varved silt, or clay with sand or silt lenses, use asphalt sand (ASTM D1073). To provide sufficient permeability to convey the seepage collected within the drainage course, several drainage layers of increasing coarseness and permeability may be required. Each layer must meet filter requirements with respect to the outer material.

b. Pipe Protection. Except for special cases such as underfloor drains, which have positive protection against the movement of soil, the drain pipe for collection of seepage should be perforated with closed joints. Discharge sections are not perforated and are laid with closed joints. Circular perforations generally should not be larger than 1/4 in. The D_{85} size of filter material shall be equal to or larger than the perforations as required in Figure 8-8. The larger values of the ratios apply to the average size of perforations. The smaller values apply to specially formed perforations where close control is exercised over hole size.

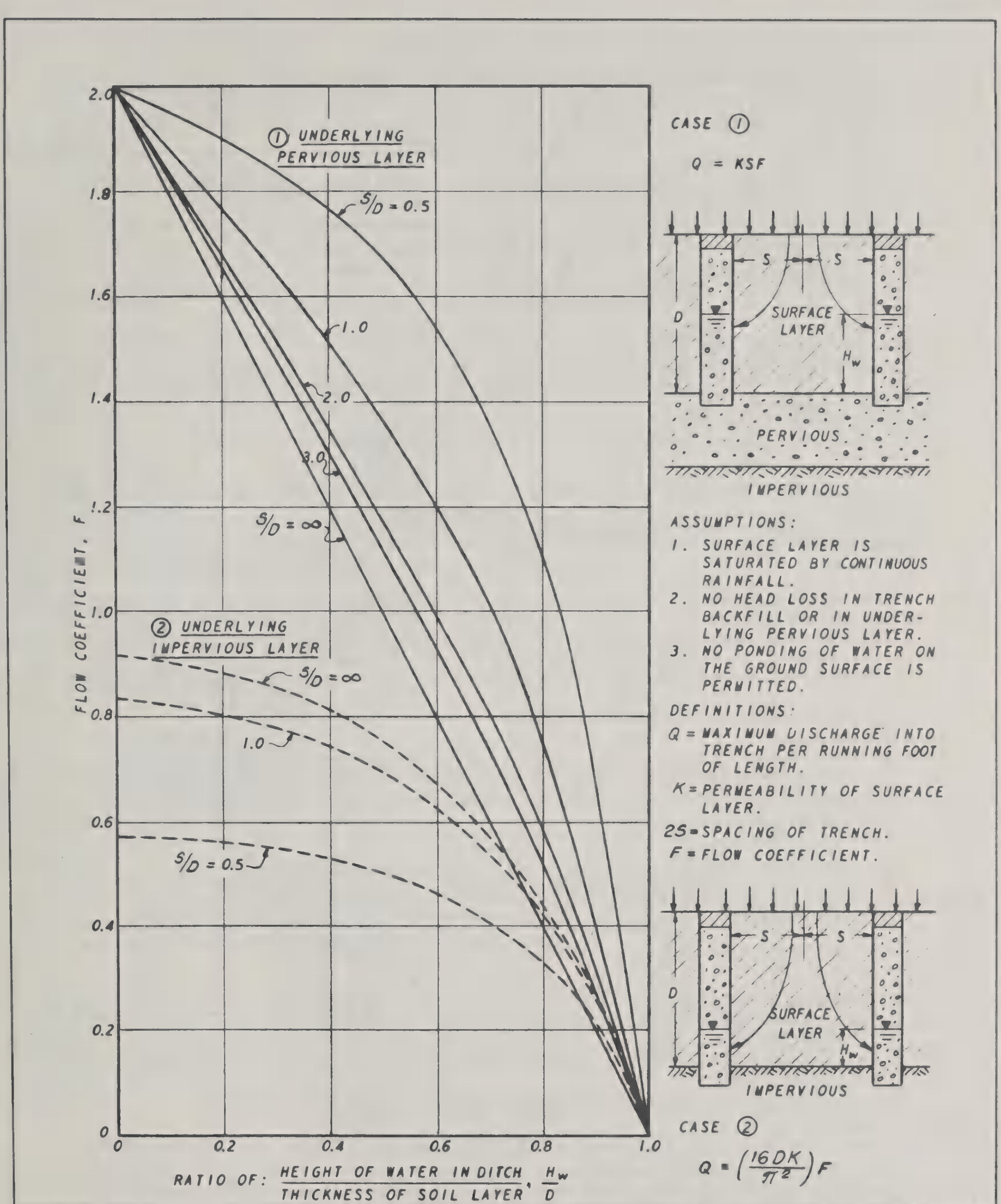
Section 6. DRAINAGE AT INTERMEDIATE DEPTHS

1. METHODS. Excavation below ground water in soils having a permeability greater than 1×10^{-3} fpm generally require drainage to permit construction in the dry. For materials with a permeability between 1×10^{-3} and 1×10^{-5} fpm, the amount of seepage may be small but drawdown of piezometric levels may still be necessary to stabilize slopes or to prevent softening of subgrades. Drawdown for intermediate depths is normally accomplished by wellpoint systems, sheeted sumps, or in special cases, by electro-osmosis.

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.

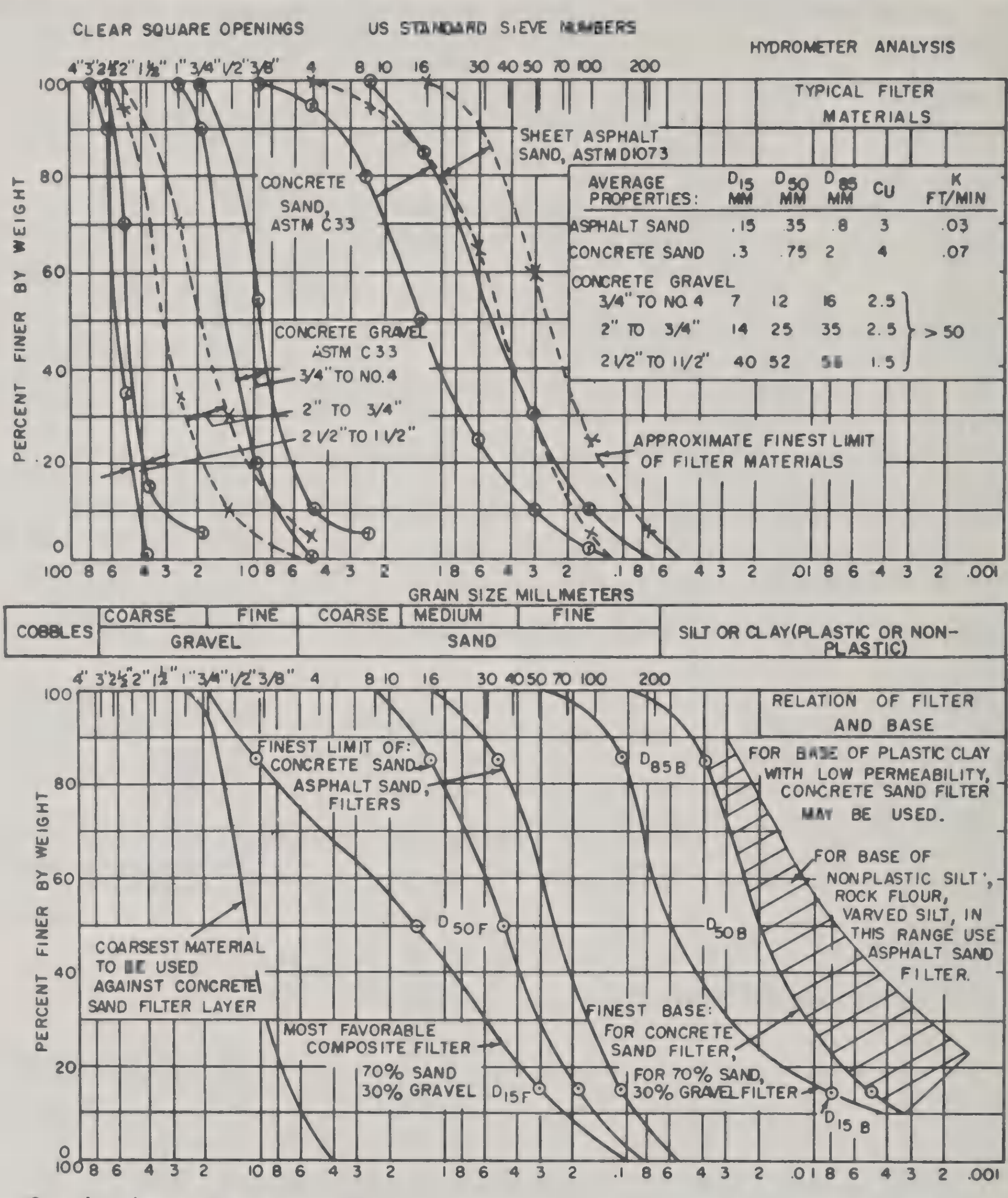
b. Settlement Effects. Where drainage will decrease pore water pressures in permeable strata adjacent to compressible soils, settlement may result. The effect of drainage on settlement of adjacent areas should be evaluated by methods of Chapter 6.

2. WELLPOINT SYSTEMS. Wellpoints consist of 1-1/2- or 2-in. 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 centrifugal pumps.



TO PREVENT PONDING OF WATER ON THE GROUND SURFACE, DESIGN DRAINAGE TRENCH SO THAT FLOW INTO TRENCH IS EQUAL OR GREATER THAN INTENSITY OF RAINFALL ON AN AREA OF: 1x2S. OTHERWISE SURFACE DRAINAGE MUST BE PROVIDED IN ADDITION TO THE TRENCHES.

FIGURE 8-7
Rate of Seepage Into Drainage Trench



General requirements:

1. To avoid head loss in filter: $\frac{D_{15\,F}}{D_{15\,B}} > 4$, and permeability of filter must be large enough to suffice for the particular drainage system.
2. To avoid movement of particles from base: $\frac{D_{15\,F}}{D_{85\,B}} < 5$, $\frac{D_{50\,F}}{D_{50\,B}} < 25$, $\frac{D_{15\,F}}{D_{15\,B}} < 20$

   For very uniform base material ($C_u < 1.5$): $D_{15\,F}/D_{85\,D}$ may be increased to 6

   For broadly graded base material ($C_u > 4$): $D_{15\,F}/D_{15\,B}$ may be increased to 40
3. To avoid movement of filter in drain pipe perforations or joints:

   $D_{85\,F}$/slot width > (1.2 to 1.4) $D_{85\,F}$/hole diameter > (1.0 to 1.2)
4. To avoid segregation filter should contain no sizes larger than 3".
5. To avoid internal movement of fines, filter should have no more than 5% passing No. 200 sieve.

FIGURE 8-8
Design Criteria for Protective Filters

a. Applicability. Pumping methods for gravity drainage generally are not effective in materials of which more than 25 percent is smaller 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 are drained by providing a vacuum seal at the ground surface around the wellpoint, utilizing atmospheric pressure as a consolidating force.

b. Capacity. Wellpoints ordinarily produce a drawdown between 15 and 18 ft below the center of the header. For greater drawdown, install wellpoints in successive tiers as excavation proceeds. Discharge capacity is generally 15 to 30 gpm per point. Points are spaced between 3 and 10 ft apart. In finely stratified or varved materials, use minimum spacing of points and increase their effectiveness by sanding an 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 materials 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. SHEETED SUMPS. For construction convenience or to handle a large flow in pervious soils, sumps are 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.

4. ELECTRO-OSMOSIS. This is a specialized procedure utilized in silts and clays that are too fine grained to be effectively drained by gravity methods. See Chapter 15 for applications.

Section 7. DEEP DRAINAGE

1. METHODS. 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 and when the principal source of seepage is from lower permeable strata.

2. 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. See Figure 8-4 for typical cross-section. Pumps may be either the turbine type with motor at the surface and pipe column with pump bowls hung inside the well, or submersible pump placed within the well.

a. Applications. Deep pumping wells are used if dewatering installations must be kept outside the excavation area, if large quantities will be pumped for the full construction period, and if pumping must commence before excavation to obtain the necessary time for drawdown. See Figure 8-9 for analysis of drawdown and pumping quantities for single wells or a group of wells in a circular pattern.

b. Special Methods. Ejector or eductor pumps may be utilized within wellpoints for lifts up to about 60 ft. The ejector or eductor pump is a nozzle arrangement at the bottom of two small diameter riser pipes placed in a wellpoint. High vacuum is created by flow through the tapered constriction. The method is useful if it is inconvenient or impossible to install a multistage wellpoint system and if the large pumping capacity of deep wells is not required.

3. RELIEF WELLS. These wells are used to bleed water from underlying strata containing artesian pressures to reduce uplift forces at a critical location. Relief wells may be tapped below ground by a collector system to reduce back pressures acting in the well.

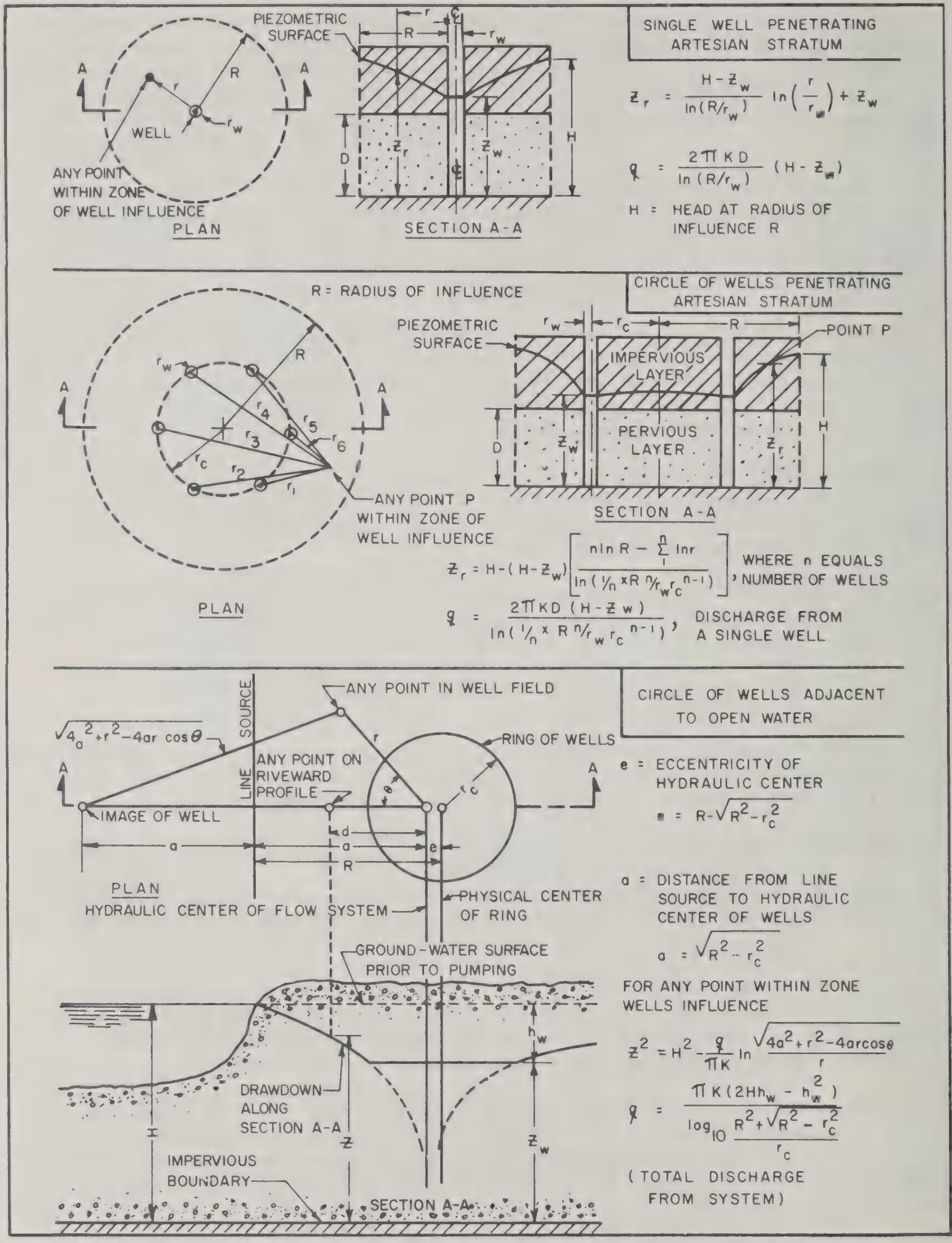


FIGURE 8-9
Ground Water Lowering by Pumping Wells

a. Applications. Relief wells are useful as a construction expedient when excavation approaches an artesian layer difficult to reach with wellpoints. 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 below the foundation. For these materials, relieve high pressures at shallow depths below the drainage course by sand drains connected to the drainage course.

b. Analysis. See Figure 8-10 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.

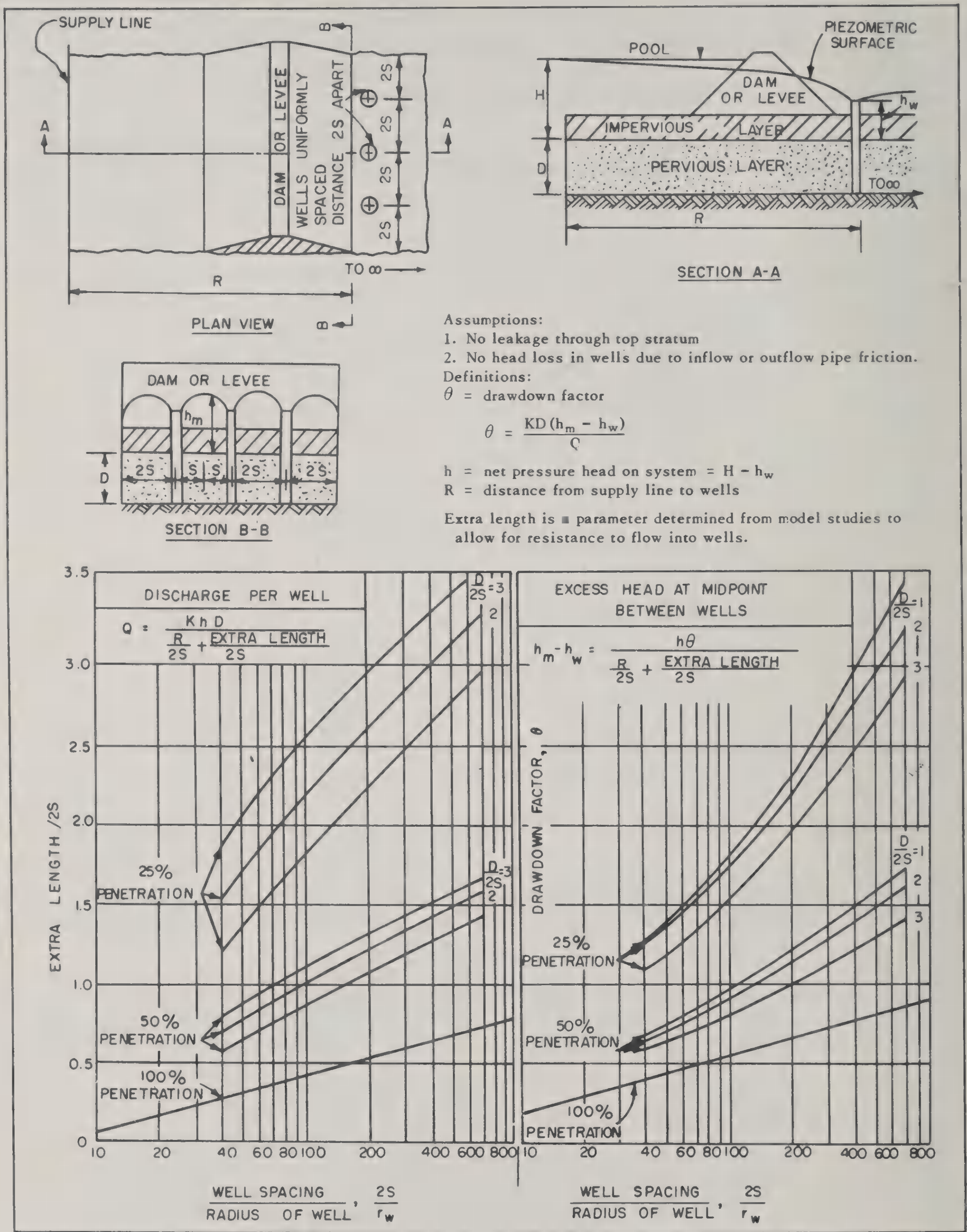


FIGURE 8-10
Drainage of Artesian Layer by Line of Relief Wells

CHAPTER 9. COMPACTED EMBANKMENTS, COMPACTION PROCEDURES, AND HYDRAULIC FILLS

Section 1. INTRODUCTION

1. **SCOPE.** This chapter concerns design and construction of compacted embankments and performance of fill materials. Compaction requirements are given for various purposes with typical equipment utilized. Earthwork control procedures and analysis of control test data are discussed.

2. **RELATED CRITERIA.** For additional criteria concerned with compaction and earthwork operations, consult the following sources:

Subject	*Source*
Compaction for airfield pavements	NAVFAC DM-21
Compaction for highway pavements	NAVFAC DM-5
Dredging equipment	NAVDOCKS DM-38
Dredging procedures	NAVFAC DM-26
Erosion control of graded surfaces	NAVFAC DM-5
Protection of riverbank or shoreline slopes	NAVFAC DM-26

3. **APPLICATIONS.** The principal uses of compacted fill include support of structures or pavements, embankments for water retention or for lining reservoirs and canals, and backfill surrounding structures or buried utilities.

Section 2. EMBANKMENT CROSS-SECTION DESIGN

1. **INFLUENCE OF MATERIAL TYPE.** See Table 9-1 for typical properties of compacted materials for use in preliminary analysis only. For final analysis perform enough tests to determine structural properties of the specific material.

a. Utilization. Practically any inorganic soil may be incorporated in an embankment when modern compaction equipment and control standards are employed. Optimum design makes the most economical use of available borrow.

b. Difficulties. The following soil characteristics or in situ conditions, interfere with economical utilization:

(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 applied pressures.

(3) Plastic soils with high natural moisture are difficult to process to proper moisture for compaction.

(4) Natural stratification may necessitate extensive mixing of borrow.

2. **EMBANKMENTS ON STABLE FOUNDATION.** The side slopes of fills not subjected to seepage forces ordinarily vary between 1 vertical to 1-1/2 horizontal and 1 to 1-3/4. Where outer sections are rock fill, slopes approach angle of repose, ranging between about 1 to 1 and 1-1/3. Details of slope profile

TABLE 9-1
Typical Properties of Compacted Materials

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: ϕ (Effective stress envelope) degrees	Typical strength characteristics: Tan ϕ	Typical coefficient of permeability ft/min.	Range of CBR values	Range of subgrade modulus k lb/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	5×10^{-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	2×10^{-6}	....	
SC	Clayey sands, poorly graded sand-clay mix.	105 - 125	19 - 11	1.1	2.2	1550	230	31	0.60	5×10^{-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	5×10^{-7}	....	
CL	Inorganic clays of low to med. 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	5×10^{-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.

and arrangement of berms are governed by requirements for erosion control. For fills supporting pavement, place coarsest soils at embankment top.

3. EMBANKMENTS ON SOFT FOUNDATIONS. Weak cohesive foundation soils may require partial or complete removal or flattening of embankment slopes. Analyze cross-section stability by methods of Chapter 7. See Chapter 6 for procedures for decreasing foundation settlement or accelerating foundation consolidation. See Chapter 15 for stabilization by densification or injection.

4. EMBANKMENT SETTLEMENT. Crest settlement is derived from three sources; foundation consolidation, embankment consolidation from dissipation of construction pore pressures, and secondary compression and shear strain in embankment after its completion.

a. Foundation Consolidation. Determine consolidation magnitude and time rate for saturated, fine-grained strata by methods of Chapter 6.

b. Loess Foundations. Unsaturated loessial silt or fine sand may settle excessively from saturation by seepage. Potential settlement depends on natural moisture and unit weight. The following table gives the lowest stable dry unit weight of silty loess at various natural moisture contents.

Natural moisture	Dry unit weight (pcf)
7% Dry of optimum	95
At optimum	86
7% Wet of optimum	75

c. Embankment Consolidation. Construction pore pressures may develop in fills exceeding about 80 ft in height or for lower heights of plastic materials placed wet of optimum moisture. Dissipation of construction pore pressures after embankment completion will cause continuing settlement. Compute maximum construction pore pressures by formula of Table 7-3, Panel (2).

d. Secondary Compression. Provide longitudinal camber of crest as parabolic curve with maximum ordinate at maximum section, equal to 1.5 times expected postconstruction settlement. For well-compacted embankment with crest width less than one-fifth of embankment heights, secondary compression and shear strain in embankment cause crest settlement after embankment completion. This amounts to between 0.1 and 0.2 percent of fill height in three to four years and from 0.3 to 0.6 percent in 15 to 20 years. The larger values are for fine grained plastic soils. Settlements similar to the maximum values occur in wetted rock fill spread in layers. Settlements similar to the minimum values occur at the centerline of embankments whose crest width is greater than one-fifth of the height.

5. EARTH DAM EMBANKMENTS. Evaluate stability at three critical stages; the end of construction, steady state seepage, and rapid drawdown. See Table 7-3 for pore pressure distribution at these stages. Requirements for seepage cutoff and stability dictate design of cross section and utilization of borrow materials.

a. Seepage Control. Place least pervious, fine grained soils in central zone and coarsest, most stable material in shells.

(1) *Cutoff Trench.* Consider the practicability of a positive cutoff trench beneath the embankment extending to impervious strata.

(2) *Intercepting Seepage.* For properly constructed zoned earth dam, there is little danger from seepage through the embankment. Drainage design generally is dictated by necessity for intercepting seepage through foundation or abutments.

(3) *Homogeneous Fills.* Downstream seepage conditions are more critical for homogeneous fills. See Chapter 8 for drainage and filter requirements.

b. Piping and Cracking. The greatest danger to low earth dams may be the threat of cracking and piping. See Table 9-2 for classification of materials according to resistance to piping or cracking. Compact the materials in lowest resistance categories to high densities with moisture content at or above optimum. Provide zoning and drainage so that seepage will not break out on downstream slope. Place internal drainage layer immediately downstream of core to control seepage from possible cracking if foundation settlements are expected to be high.

Section 3. COMPACTION REQUIREMENTS AND PROCEDURES

1. COMPACTION REQUIREMENTS.

a. Summary. See Table 9-3 for a summary of compaction requirements of fills for various purposes. Modify details to meet conditions and materials of specific projects.

b. Specification Provisions. Specify either the method of compaction or the desired compaction result. In any case, state maximum tolerable lift thickness. Where a test section has been completed or where equipment performance on a material is known, specify compaction method and provide for plus or minus unit price for change in number of coverages that proves to be necessary to achieve desired density. When alteration of unit price is to be avoided, specify minimum compactive effort to achieve disired density and require contractor to increase coverages as necessary to reach desired sensity. Otherwise state required density, moisture limits, and lift thickness, allowing the contractor some selection in compaction methods.

2. COMPACTION METHODS. See Table 9-4 for summary of compaction equipment with typical sizes and weights used. Details given are an approximate guide for equipment selection. For specific materials, verify the performance of equipment by test sections or vary coverages as necessary during construction.

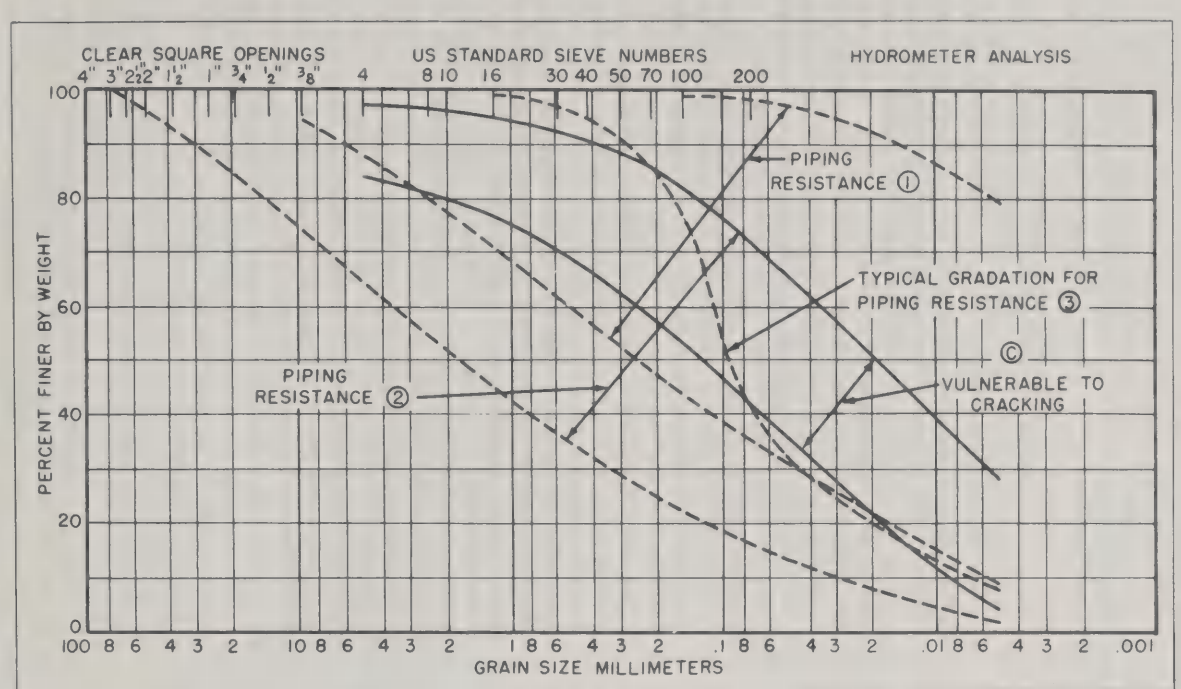
3. INFLUENCE OF MATERIAL TYPE. See Table 9-4 for applicability of material type.

a. Soils Insensitive to Compaction Moisture. Coarse grained, cohesionless soils with less than 4 percent passing No. 200 sieve for well-graded soils, or with less than 8 percent for uniform gradation, are insensitive to compaction moisture. These soils have a permeability greater than about 2×10^{-3} fpm. Place these materials at the highest practical moisture content, preferably saturated. Compaction by vibratory methods generally is the most effective procedure. In these materials 70 to 75 percent relative density is obtained by proper compaction methods. If 70 to 75 percent relative density is substantially higher than standard Proctor maximum density, use relative density test for control. See Table 3-3 for test procedure.

b. Soils Sensitive to Compaction Moisture. For ordinary mass earthwork with fine grained or dirty, coarse grained soils, use standard Proctor test, ASTM D698, for control. Apply modified Proctor test for compaction of airfield pavement base course and subgrade only.

c. Effect of Oversize. Where materials with sizes up to 6 in. maximum are used, the large sizes will interfere with compaction of soil smaller than No. 4 sieve or 3/4 in. (fractions which are used in the density test). For mass earthwork, a reduction in densities of the fraction smaller than No. 4 sieve or 3/4 in.

TABLE 9-2
Resistance of Earth Dam Embankment Materials to Piping and Cracking



COBBLES	COARSE	FINE	COARSE	MEDIUM	FINE	SILT OR CLAY (PLASTIC OR NON-PLASTIC)
	GRAVEL		SAND			

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.
Cracking resistance:		
Ⓐ	CH with D_{50} < 0.02mm and PI > 20.	High postconstruction settlement, particularly if compacted dry. Has sufficient deformability to undergo large shear strains from differential settlement without cracking.
Ⓑ	GC, SC, SM, SP with D_{50} > 0.15mm.	Small postconstruction settlement. Little chance for cracking unless poorly compacted and large settlement is imposed on embankment by consolidation of the foundation.
Ⓒ	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.

TABLE 9-3
Compaction Requirements

Fill utilized for:	Required density, percent of standard Proctor	Tolerable range of moisture about optimum, percent	Maximum permissible lift thickness, compacted in.	Special requirements
Support of structure . .	95	− 2 to + 2	8	Fill should be as uniform as possible and 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 but no higher than necessary.
Lining for canal or small reservoir.		− 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 in. before compaction.
Earth dam greater than 50 ft high.	98	− 1 to + 2	8	Utilize least pervious materials as central core and coarsest 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. Smaller amounts can be removed by hand picking or by rock rakes.
Earth dam less than 50 ft high.	95	− 1 to + 3	8	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 coarsest 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. See NAVFAC DM 5 for requirements for compaction of highway pavement subgrade and NAVFAC DM 21 for requirements for compaction of airfield pavement subgrades.
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 ma- 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 high walls or walls of special design, evaluate the surcharge produced by heavy compaction equipment by the methods of Chapter 10 and specify safe distances back of the wall for its operation.

TABLE 9-3 (Continued)
Compaction Requirements

Fill utilized for:	Required density, percent of standard Proctor	Tolerable range of moisture about optimum, percent	Maximim 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 flowed saturated under and around the pipe and compacted by vibration.
Drainage blanket or filter.	95	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, blasting, or pile driving.
Rock fill............		Thoroughly wetted	2 to 3 ft.	For fill containing sizes no larger than 1 ft, place in layers not exceeding 24 in., thoroughly wetted and compacted by travel of 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 "standard Proctor" test values, (ASTM D 698).
2. "Coarse, Cohesionless Soil" has less than 4 percent by weight passing the No. 200 sieve for well-graded soil with C_u greater than 6, and less than 8 percent by weight passing the No. 200 sieve for narrowly graded soil with C_u less than 3. These soils are not sensitive to compaction moisture content so long as bulking moisture is avoided. Where practicable, they should be placed saturated and compacted by vibratory methods.

TABLE 9-4
Compaction Equipment and Methods

Equipment type	Applicability	Requirements for compaction of 95 to 100 percent standard Proctor maximum density: Compacted lift thickness, in.	Passes or coverages	Dimensions and weight of equipment	Possible variations in equipment
Sheepsfoot rollers.	For fine grained soils or dirty coarse grained soils with more than 20 percent passing the 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 in. — 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 Effecient compaction of soils wet of optimum requires less contact pressures than the same soils at lower moisture contents.	For earth dam, highway, and airfield work, drum of 60-in. dia., loaded to 1.5 to 3 tons per lineal ft of drum generally is utilized. For smaller projects 40-in.-dia. drum, loaded to 0.75 to 1.75 tons per lineal ft of drum is used. Foot contact pressure should be regulated so as to avoid shearing the soil on the third or fourth pass.
Rubber tire rollers.	For clean, coarse grained soils with 4 to 8 percent passing the No. 200 sieve.	10	3 to 5 coverages.	Tire inflation pressures of 60 to 80 psi for clean granular material or base course and subgrade compaction. Wheel load 18,000 to 25,000 lb.	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 cohesionless 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 [illegible]	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 lb per lineal in. of width of rear roller.	3-wheel rollers obtainable in wide range of sizes. 2-wheel tandem rollers are available in the range of 1 to 20 ton weight. 3-axle tandem rollers are generally used in the range of 10 to 20 ton 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 9-4 (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 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, placed thoroughly wet.	8 to 10	3 coverages.	Single pads or plates should weigh no less than 200 lb. 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 or self-propelled, single or in gangs, with width of coverage from 1½ 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.	10 to 12	3 to 4 coverages.	No smaller than D8 tractor with blade, 34,500 lb weight, for high compaction.	Tractor weights up to 60,000 lb.
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 lb, foot diameter 4 to 10 in.

equal to several percent of maximum density, usually is tolerable where a large proportion of gravel and cobbles is present.

Section 4. EMBANKMENT COMPACTION CONTROL

1. REQUIREMENTS FOR CONTROL TESTS. Perform field density tests plus sufficient laboratory moisture-density tests to evaluate compaction achieved. For high embankments involving seepage, settlement, or stability, perform periodic tests of structural properties on density test samples. See Chapter 4 for field density test procedures. For laboratory test procedures, see Chapter 3.

a. Number of Field Density Tests. Schedule the following number of tests:

(1) One for every 2,000 cu yd of material placed for mass earthwork.

(2) One for every 1,000 cu yd of material in relatively thin sections for canal or reservoir lining.

(3) One for every 200 to 500 cu yd of backfill in trenches or surrounding 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 the quality of moisture control or effectiveness of compaction.

b. Number of Laboratory Compaction Tests. Prior to important earthwork operations, obtain a family of compaction curves representing typical materials. Ideally, this family forms a group of parallel curves and each field density test would correspond to a specific compaction curve.

(1) Supplementary Tests. Obtain supplementary compaction curves on field density test samples, approximately one for every 10 or 20 field tests, depending on the variability of materials.

(2) Alternative Tests. As an alternative, apply a rapid compaction control test to each field density sample to determine deviation of field density from laboratory maximum. Use USBR, *Earth Manual,* Method E-25.

2. ANALYSIS OF CONTROL TEST DATA. Compare each field determination of moisture and density with appropriate compaction curve to evaluate conformance with 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 median moisture content specified. Erratic moisture control is evidenced if only two-thirds of all field values fall in a range ±2 or 3 percent about median moisture content specified. To improve moisture control, irrigate or drain borrow area well in advance of excavation, bypass overly wet borrow material, blend materials from wet and dry sections of borrow area.

c. Compactive Effort. Suitable compaction methods are being utilized if two-thirds of all field densities fall in a range of ±2 or 3 percent about the percent maximum density required. Insufficient or erratic compaction is evidenced if only two-thirds of all field values fall in a range of ±4 or 5 percent about the percent maximum density required. To improve compaction, consider methods for more uniform moisture control, alter the number of coverages or weights or pressures of compaction equipment.

Section 5. BORROW EXCAVATION

1. **EXCAVATION METHODS.** See Abbett, *American Civil Engineering Practice*, Vol. 1 for information on excavation equipment suitable for various purposes and materials.

a. **Blasting.** Determine necessity for blasting by test borings carried to lowest level of proposed excavation.

b. **Preparation by Ripping.** Certain hard and rocky soils may be prepared for excavation by surface scarification with rock rake or ripper without blasting. Evaluate rippability by shallow seismic investigation using sledge hammer blows for impulse.

2. **UTILIZATION OF EXCAVATED MATERIALS.** For maximum economy in earthwork operations, utilize for the fill all inorganic materials from required excavation.

a. **Borrow Volume.** Determine excavation required for compacted fill by Equation 9-1:

$$\text{Total borrow volume required} = \left(\frac{\text{Dry unit weight of fill}}{\text{Dry unit weight of borrow}}\right) \times (\text{Fill volume required}) + \left(\frac{\text{Weight lost in stripping, waste, oversize, or transportation}}{\text{Dry unit weight of borrow}}\right) \tag{9-1}$$

(1) Volume Increased. Fill volume required should be increased by the compression that will be experienced under final load. This value may be 1 to 2 percent of nominal volume. See Table 9-1 for typical compression values.

(2) Exclusions. For earth dam embankments, sized larger than 6 in. generally are excluded from central zones, but may be placed on the shells. For highway, airfield, or structural fills, sizes larger than 6 to 10 in. generally are excluded from soil fill that must be thoroughly compacted but may be used on outer slopes.

b. **Rock Fill.** For precise determination of swell factor for rock, perform blasting test to evaluate breakdown of sizes and place these materials in a small test section to determine fill volume.

(1) Maximum Expansion. Maximum expansion from in situ conditions to fill occurs in dense, hard rock with fine fracture system that breaks in uniform sizes. Unit volume in a quarry may produce 1-1/2 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 may produce 1.1 volumes in fill.

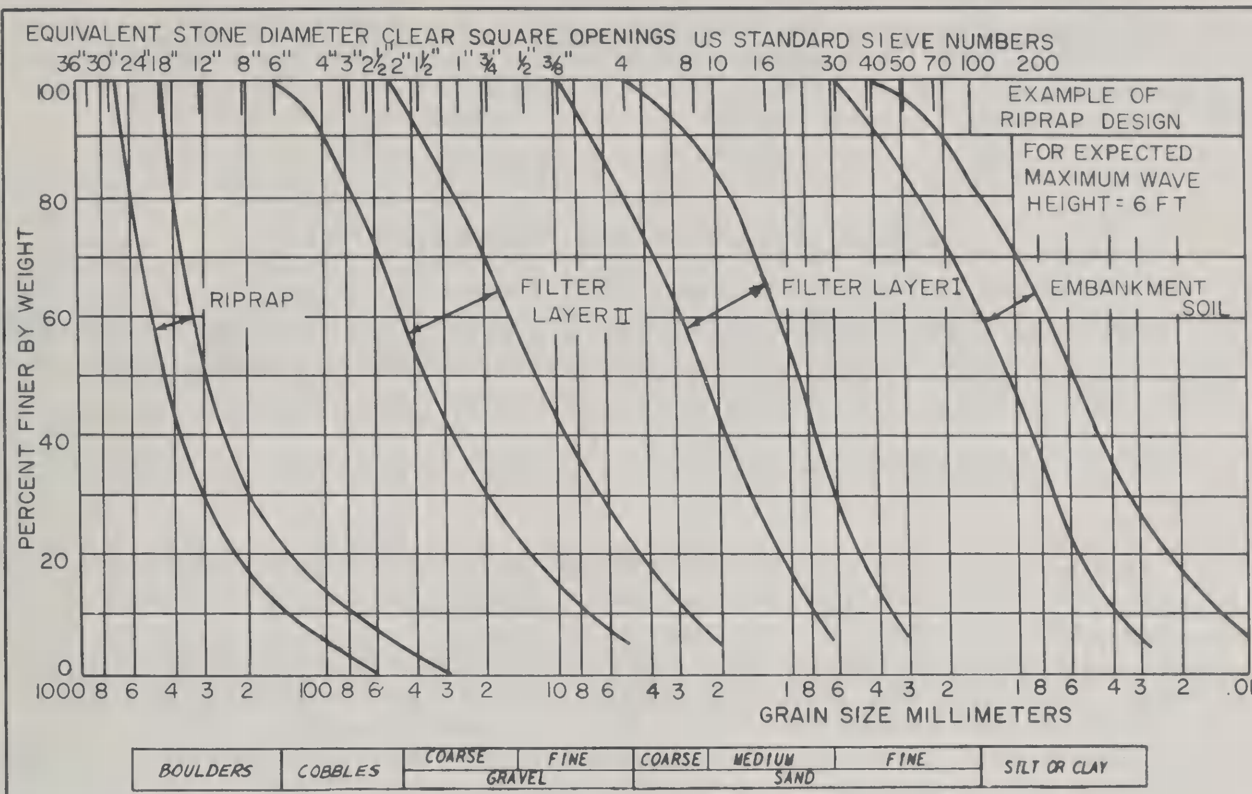
Section 6. EMBANKMENT SLOPE PROTECTION

1. **METHODS.** See Table 9-5 for principal types of protection for upstream slope of earth dams, reservoirs, or canals exposed to ordinary wave action. More elaborate protection may be required for riverbanks or shorelines subject to major erosive forces.

a. **Dumped Riprap.** This material provides slope protection of widest applicability. See Figure 9-1 for design criteria.

TABLE 9-5
Slope Protection and Lining for Earth Dams, Reservoirs, and Canals

Materials	Applicability	Characteristics and requirements
Dumped rock riprap	Most effective method of wave protection and wave energy dissipation, resistant to frost, reservoir ice, or damage by floating debris or embankment settlement.	See Figure 9-1 for design criteria. May not be economical where rock source is distant.
Hand placed rock riprap.	Satisfactory wave protection where not exposed to heavy ice conditions. Vulnerable to damage when ice and floating debris gouge individual rocks from place. Displacement of single rocks exposes bedding material and leads to progressive deterioration.	Preferably uniform size stones of square shape laid with minimum openings, chinked with spalls where necessary. Thickness of layer about two-thirds that for dumped rock riprap but not less than 12 in. Where rock source is at great distance it may be cheaper than dumped riprap.
Separate or articulated concrete slab.	Performance record as wave protection has been poor due to undermining of slabs by piping of bedding material through joints. Suitable only where wave action is minimum in small reservoirs or canals. Granular bedding layer may be required to eliminate frost heave.	Slab thicknesses range from 6 in. minimum to 8 in. for 50 ft dam; with maximum of 12 in. Reinforcement area is 0.3 to 0.5 percent of concrete area, both ways. High density, low slump, air-entrained concrete. Seal joints with durable fillers and grout or seal cracks promptly.
Monolithic concrete paving.	Used for impervious membrane on rock fill or gravel dams as well as wave protection. Cracking is inevitable, Provide drainage blanket beneath slab if seepage is to be kept out of embankment.	Slab thickness generally 1 percent of water head but not less than 8 in. for impervious membrane. Reinforcing area is 0.5 percent vertically, 0.7 percent horizontally of concrete area. Steel continuous across construction joints. High density, low slump, air-entrained concrete.
Asphaltic paving ...	Used for impervious membrane on rock fill or gravel dams as well as wave protection for fine grained embankments. Lower cost than concrete or steel. Accommodates differential settlement without cracking. Vulnerable to damage by debris or ice gouging.	Variety of mixes utilized. Hot mix asphalt is typical, 8 percent asphalt by weight in aggregate ranging from 11 percent passing No. 200 to 1-1/2 in. maximum. Total thickness vary from 6 to 12 in. In some instances, thin asphalt layers are separated by drainage layer to prevent seepage in embankment.
Asphalt panels	Prefabricated panels of asphalt-impregnated fibers used for impervious reservoir lining where wave action is minimum. Economical in proper application.	Panels about 1/2 in. thick. Lap joints or butt joints with overlay strip painted with hot asphalt adhesive. Finish fill surface with smooth roller. Anchor panels at top of slope with spikes.
Steel plate	Used for impervious membrane on rock fill or gravel dams and occasionally as wave protection for fine grained embankments. Accommodates differential settlement without rupture. High first cost compared to other membrane types, requires periodic cleaning and painting.	Thickness vary from 1/4 to 1/2 in. Sizes depend on handling equipment. Form contraction joints by vertical V-shaped troughs at about 25 ft spacing. Drain beneath contraction joints is desirable because welds may crack. Carry plate to concrete cutoff wall at toe for water-tight contact.
Brush mattresses ...	Utilized on small earth dams for erosion control and wave protection. Lowest first cost, but usually requires frequent replacement.	Saplings 1 to 2 in. diameter, up to 25 ft long assembled in bundles 12 to 18 in. diameter, tied with wire. Bundles laid on slope, butt downhill, and woven together with heavy wire or cable anchored to deadman in embankment.
Heavy compacted soil lining.	Impervious lining 3 ft wide normal to slope for canals or small reservoirs. Costs roughly one-quarter of unreinforced concrete lining. Especially suited for locations with expansive clays that may rupture thin concrete lining.	Preferably GW-GC, GC, SC. Placed in 6-in. lifts, at or above optimum moisture, compacted to at least 95 percent of "standard Proctor" maximum density (ASTM D 698).
Thin compacted soil lining.	Impervious lining 6 to 12 in. thick, for canals or small reservoirs. Usually requires gravel cover for erosion protection. Unit costs relatively high compared to thick soil lining.	Requirements similar to that for heavy compacted lining, except that CL or CH are utilized and special equipment is needed for compaction.



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 1½ IN.
4 TO 10	1½ TO 2 IN.

NO FILTER IS NEEDED IF EMBANKMENT MEETS THE ABOVE REQUIREMENTS FOR D_{85} SIZE.

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:

MAXIMUM WAVE HEIGHT, FT	FILTER THICKNESS, IN.
0 TO 4	6
4 TO 8	9
8 TO 12	12

DOUBLE FILTER LAYERS SHOULD BE AT LEAST 6 IN. THICK.

FIGURE 9-1

Design Criteria for Riprap on Earth Embankments

b. Impervious Linings. Certain types of upstream slope protection also constitute impervious linings for use on fills of rock or coarse-grained soil to prevent embankment seepage. If an impervious membrane is used instead of a central impervious zone, consider providing a drainage blanket or collectors beneath the membrane to control leakage that could result from cracking of the membrane. See Table 8-2 for methods of impermeabilizing small reservoirs where wave action is minor.

Section 7. HYDRAULIC AND UNDERWATER FILLS

1. TYPES. Hydraulic fills are placed on land or under water by pumping material through a pipeline or by water sluicing through a conveyor. Borrow materials generally are obtained by dredging. Underwater fills may be placed by hydraulic methods or by dumping dry borrow through water.

2. CONSTRUCTION METHODS. The choice of construction method depends on the location of the fill.

a. Requirements for Hydraulic Fill on Land. Wash water should run off in such a manner that fines are not concentrated in pockets. This may require advancing the fill from one side or corner of the area, attempting to force out any soft fines ahead of the fill as it is placed. To estimate settlement of coarse grained hydraulic fill, determine relative density from standard penetration resistance and use correlations of Chapter 3. For detailed study, perform consolidation tests on representative samples placed at a void ratio equivalent to field condition. For uniform bearing directly below the foundation, recompact a blanket of upper materials in the dry. For special requirements for hydraulic fills placed behind walls or bulkheads, see Chapter 10.

b. Requirements for Underwater Fills. For structural fill placed on a dredged bottom in soft material, remove fines dispersed in dredging, by a final sweeping operation by suction dredges before placing fill. To prevent extremely flat slopes at the edge of a fill, avoid excessive turbulence in dumping fill material. Uncontrolled bottom dumping from barges through great depths of water will spread the fill over wide area. To confine the fill, provide berms or dikes of the coarsest material or stone on the periphery of fill area.

3. PERFORMANCE OF FILL MATERIALS. Ordinarily, hydraulic or underwater fills for support of structures are formed of the coarsest material economically available.

a. Coarse Grained Fills. Unless special provisions are made for removal of fines by sluicing, avoid using borrow with more than 15 percent nonplastic fines or 10 percent plastic fines passing No. 200 sieve. To obtain relatively steep slopes in underwater fill, use mixed sand and gravel. With borrow containing about equal amounts of sand and gravel, underwater slopes as steep as 1 vertical to 3 horizontal or 1 to 2-3/4 may be achieved by careful placement.

b. Fills of Hard Clay. Hydraulically placed stiff to hard clay, excavated by suction dredge with cutterhead, produces a fill of boulderlike clay clods if fines in the wash water are permitted to run off. Slopes of such fills will be extremely flat, ranging from about 12 to 16 horizontal to 1 vertical. These fills will undergo large immediate consolidation for about the first six months until the clay clods distort to close void spaces. Additional settlements for a 10-year period after this time will total about 3 to 5 percent of fill height. Long-term consolidation may be reduced by placing surcharge before paving or construction.

CHAPTER 10. 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 DM volumes:

Subject	*Source*
Application of bulkheads and cofferdams to waterfront construction	NAVFAC DM-25 and NAVFAC DM-26
Structural design and details of bulkheads and cofferdams	NAVFAC DM-25
Structural design of retaining walls	NAVFAC DM-2

Section 2. COMPUTATION OF WALL PRESSURES

1. ACTIVE PRESSURES. The intensity of pressures applied depends on wall movements, which control the degree of shear strength mobilization in surrounding soil. Active values are minimum earth pressures tending to move the wall outward.

a. Uniform Backfill, No Ground Water. Compute active pressures by methods of Chapter 5.

b. Uniform Backfill, Static Ground Water. Compute active earth and water pressures by formulas in Figure 10-1.

c. Stratified Backfill, Sloping Ground Water Level. When conditions include layered soil, irregular surcharge, wall friction, and sloping ground water level, determine active pressures by trial failure wedge. (See Figure 10-1.) 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 5-12.

(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.

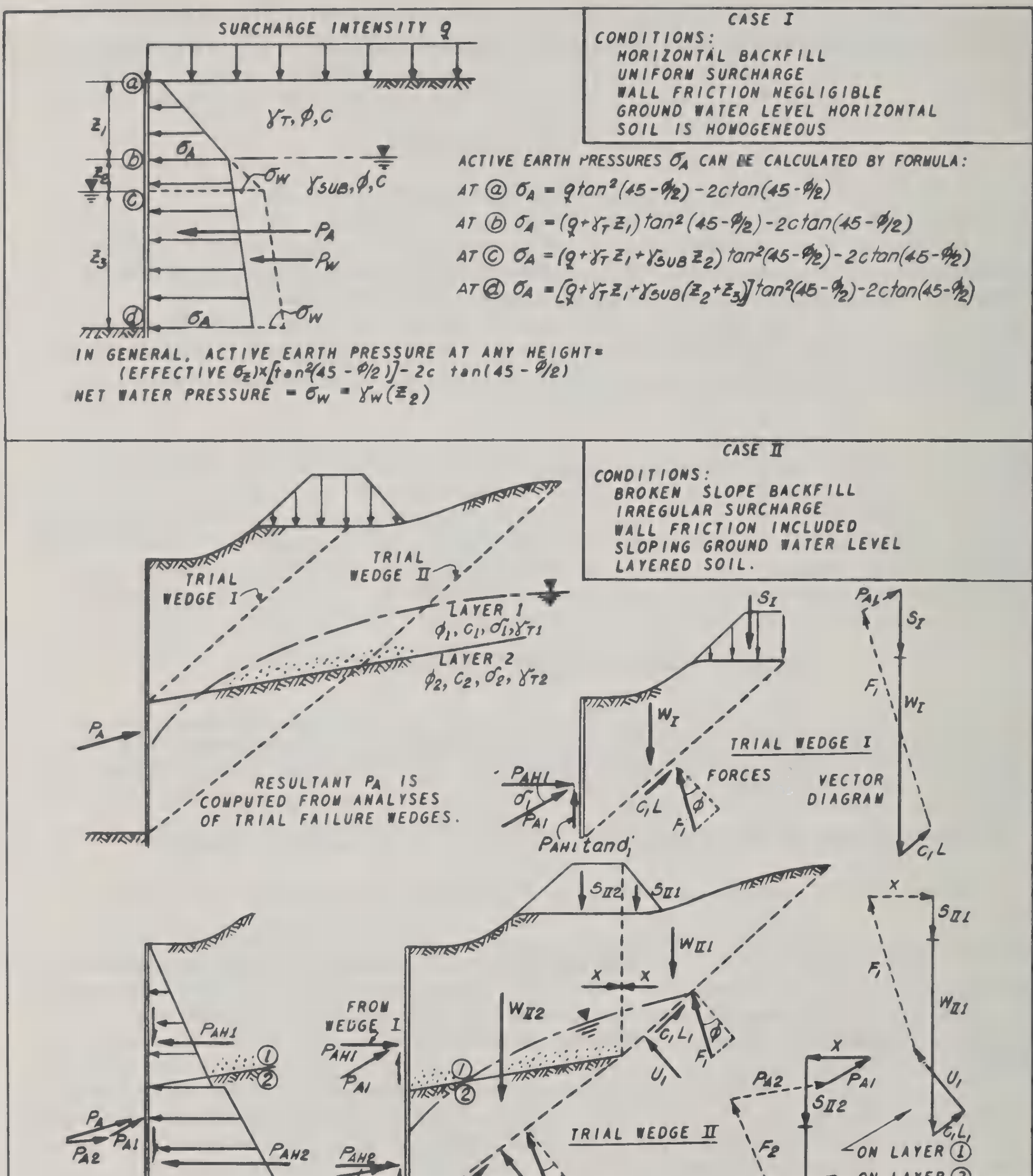


FIGURE 10-1
Computation of General Active Pressures

2. **PASSIVE PRESSURES.** Where movement fully mobilizes shear strength, passive earth pressures are maximum values available to resist failure.

a. **Uniform Backfill, No Ground Water.** Compute passive pressures by methods of Chapter 5.

b. **Uniform Backfill, Static Ground Water.** Compute passive earth and water pressures by formulas in Figure 10-2.

c. **Stratified Backfill, Sloping Ground Water Level.** For more complicated cross sections, compute resultant passive force by trial failure wedge analysis. (See Figure 10-2.) 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 5-10.

3. **PRESSURE COEFFICIENTS WITH WALL FRICTION.** For simple cross sections in cohesionless materials where wall friction is included, use active and passive coefficients of Figures 10-3 and 10-4.

a. **Direction of Wall Friction.** Unless a wall is settling, friction on its back acts upward on the active wedge (angle δ is positive), reducing active pressures. Generally, wall friction acts downward against the passive wedge (angle δ is negative), resisting its upward movement and increasing passive pressures. The effect of wall friction on active pressures is small and ordinarily is disregarded. The effect of wall friction on passive pressures is large, but definite movement is necessary for mobilization of wall friction. Ordinarily a safety factor of at least 2 against passive failure is required when wall friction is included.

b. **Wall Friction Factors.** See Table 10-1 for typical 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.

4. **EFFECT OF SEEPAGE AND DRAINAGE.** Include in pressure computations the effect of the greatest unbalanced water head that will 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 in Figures 10-1 and 10-2. For approximate analyses, take uplife intensity equal to pressure of the vertical height of water between ground water surface and a point directly beneath on failure surface.

b. **Static Differential Head.** For the effect on wall pressures, see top panel of Figure 10-5. For static differential head with insignificant seepage, compute water pressures on walls by formula, Figure 10-1. In cohesionless soils, the active force on a wall with static water level at the top of backfill is double that for dry backfill.

c. **Rainfall on Drained Walls.** The center panel of Figure 10-5 shows flow net set up by rainfall behind a wall with vertical drain. For cohesionless materials, sustained rainfall increases active force 20 to 40 percent over dry backfill, depending on backfill friction angle. This panel gives the magnitude

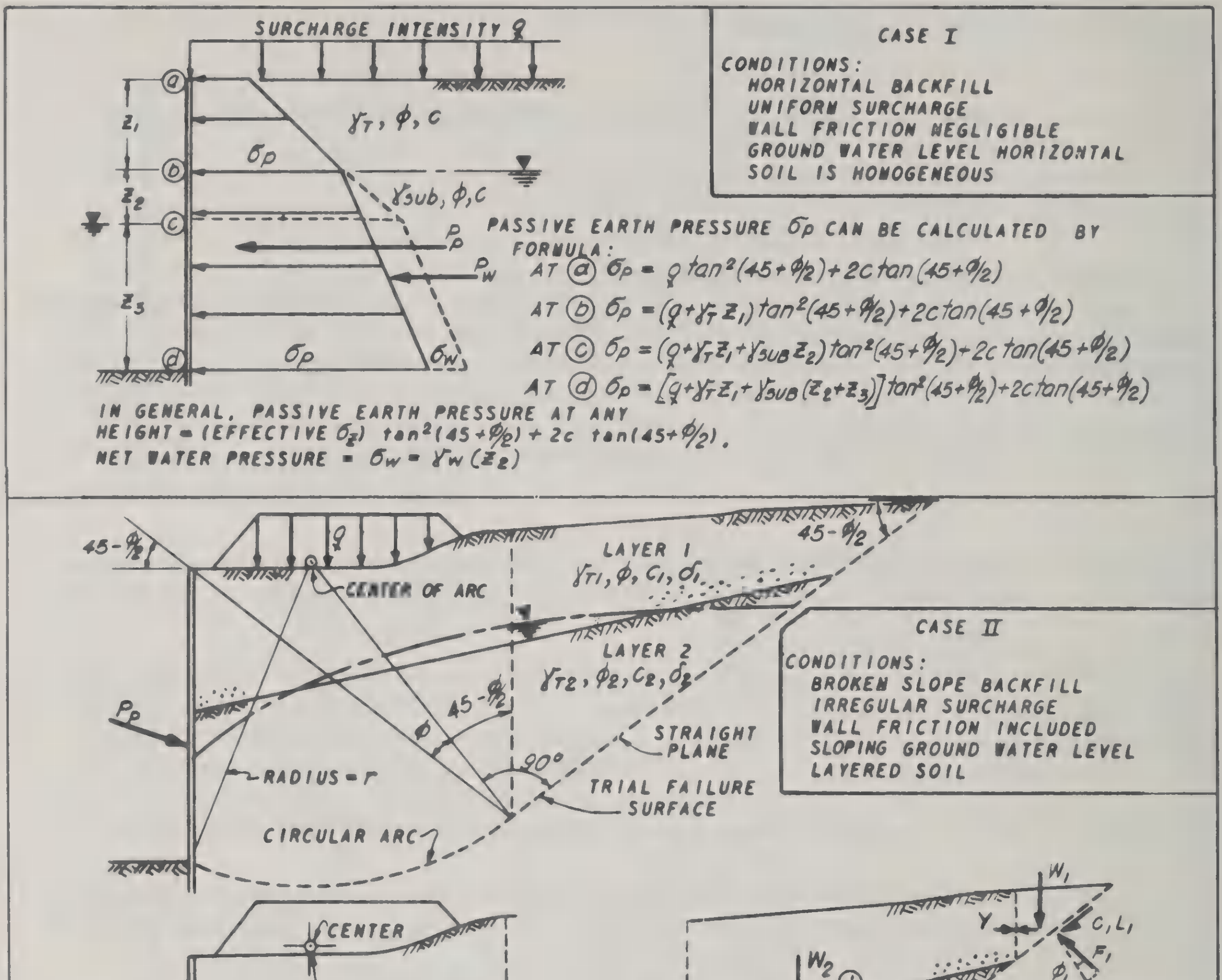
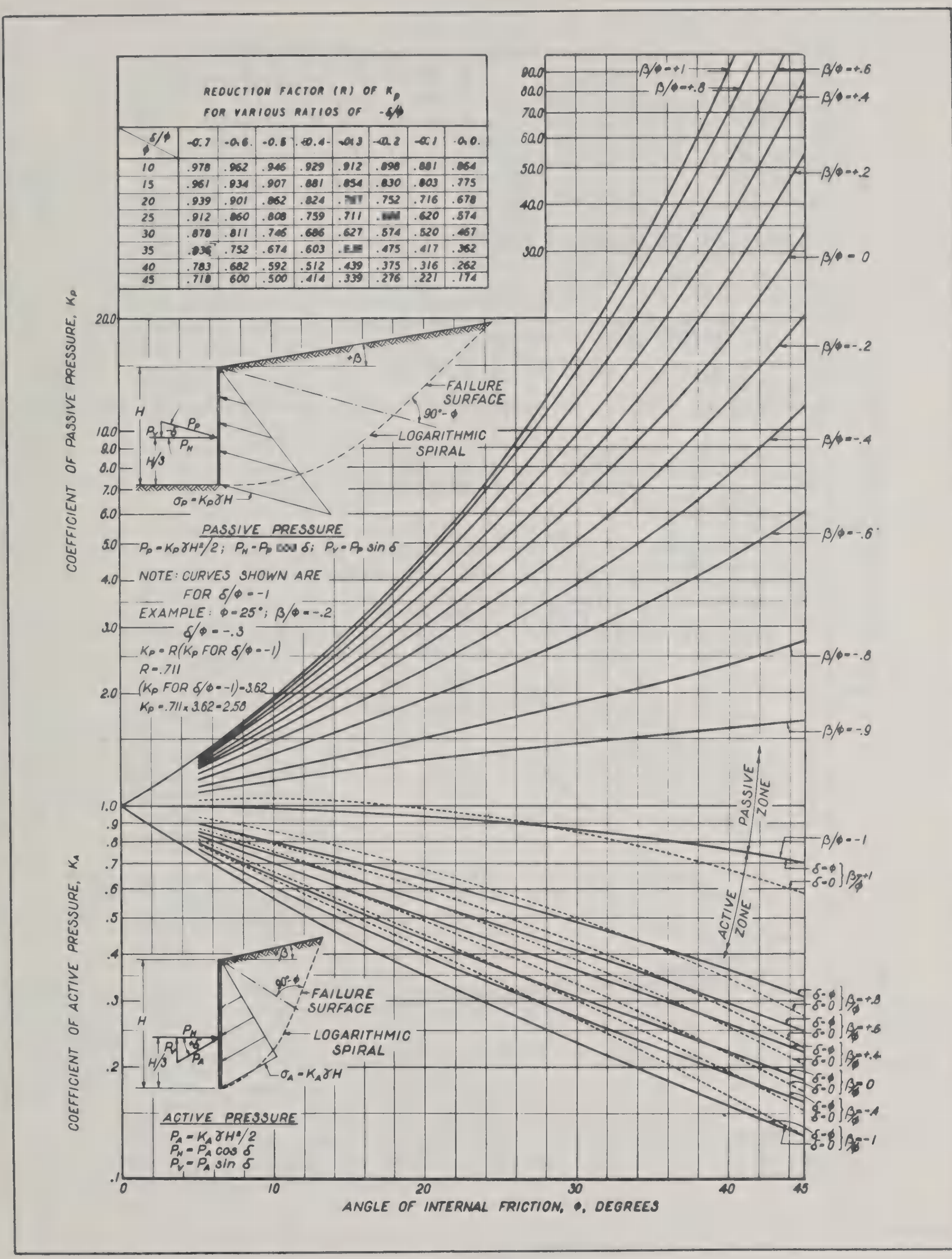


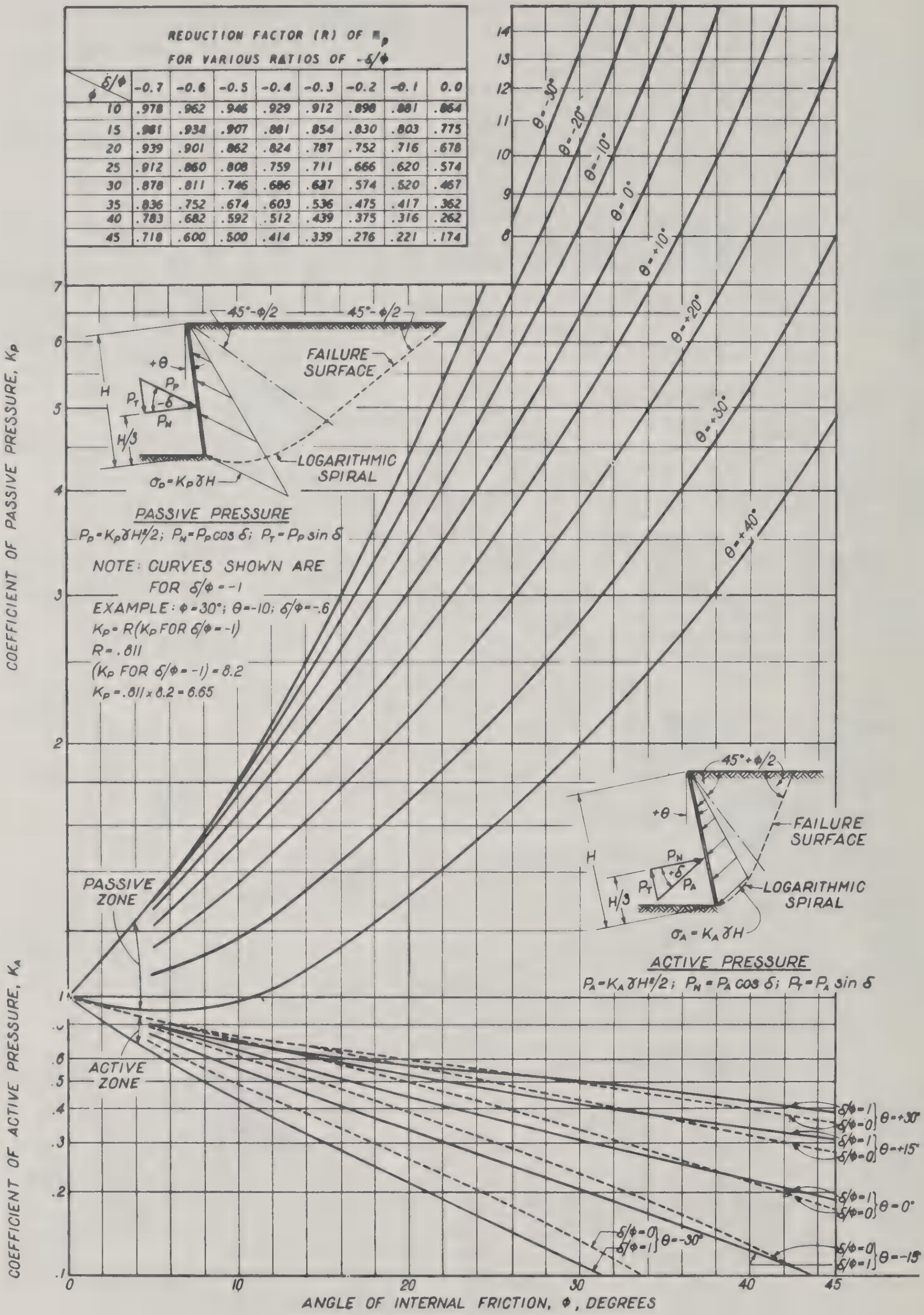
FIGURE 10-2
Computation of General Passive Pressures



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	[illegible]	.752	.716	.678
25	.912	.860	.808	.759	.711	[illegible]	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	[illegible]	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	600	.500	.414	.339	.276	.221	.174

FIGURE 10-3
Active and Passive Coefficients with Wall Friction (Sloping Backfill)



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	.981	.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 10-4
Active and Passive Coefficients with Wall Friction (Sloping Wall)

TABLE 10-1
Friction Factors and Adhesion for Dissimilar Materials

Interface materials	Friction factor, tan δ	Friction angle, δ, degrees	Adhesion C_A, psf
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	
Soft clay and clayey silt			100 to 600
Stiff and hard clay and clayey silt			600 to 1,200
Formed concrete or concrete sheet piling against the following soils:			
Clean gravel, gravel-sand mixtures, 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	
Soft clay and clayey silt			200 to 700
Stiff and hard clay and clayey silt			700 to 1,200
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	

Note: Numbers shown are ultimate values and require sufficient movement for failure to occur.
Where friction factor only is shown, the effect of adhesion is included in the friction factor.
For data on adhesion on bearing piles, see Chapter 13.

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 10-5 for correction to be applied to active and passive pressures in cohesionless material for steady seepage beneath a wall.

e. Consolidation Pore Pressures. Where impervious clay behind a wall is loaded by backfill, hydrostatic excess pore pressures can develop in the clay. Estimate such pore pressures by methods of Chapter 6 and include them in uplift on the failure surface.

5. SURCHARGE LOADING. For the effect of surcharge loading, see Figures 10-1 and 10-2.

a. Area Loads. Where surcharge behind a wall consists of area load, include the weight of surcharge with forces acting on the trial failure wedges.

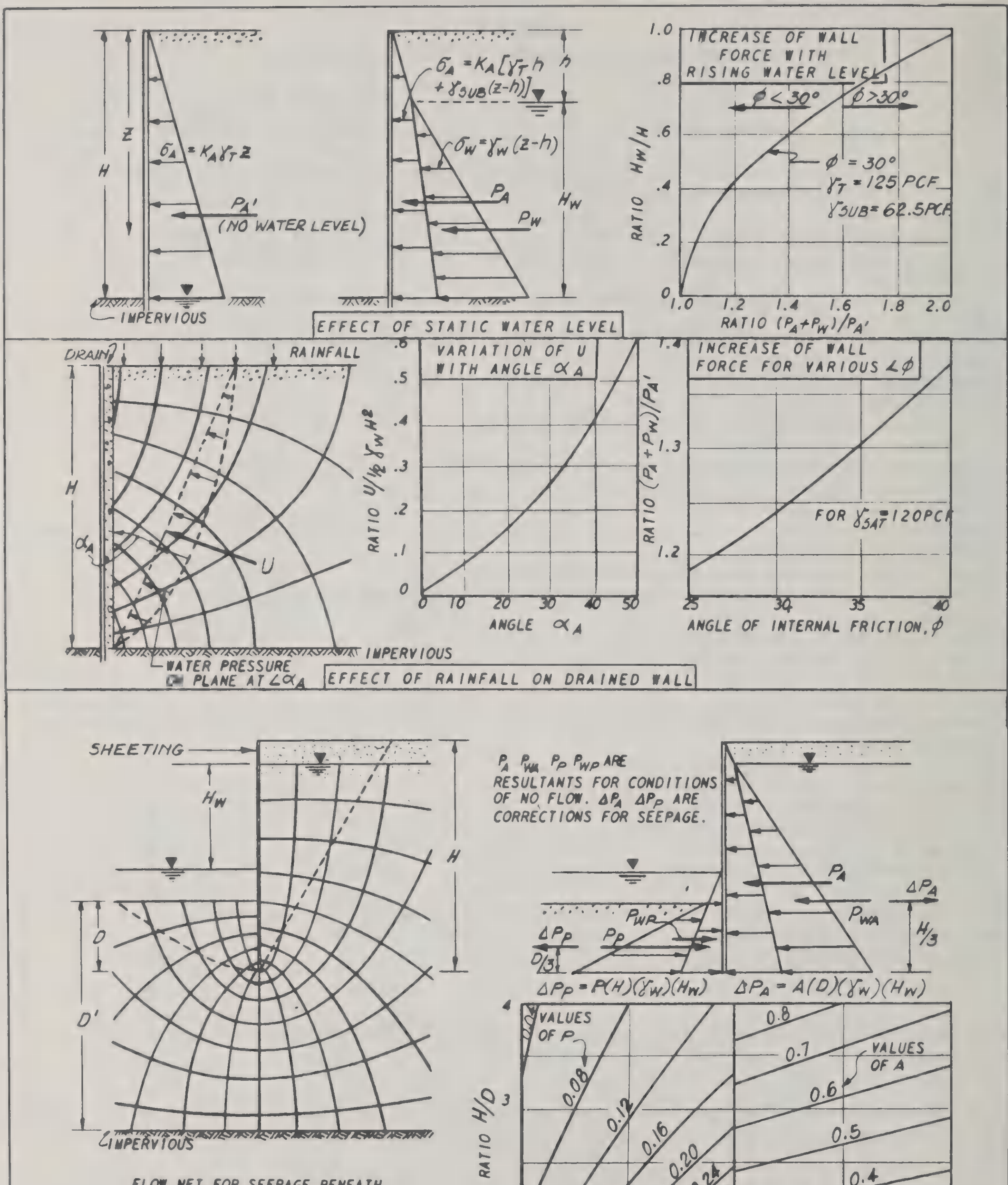


FIGURE 10-5
Effect of Ground Water Conditions on Wall Pressures

b. Live Loads. Compute wall pressures from the point or line load, whose intensity is small compared to backfill pressures, by formulas of Figure 10-6. These horizontal pressures are approximately double the values derived from elastic equations. A wall is assumed to have undergone its principal movement before placing surcharge and to be unyielding during the application of live load.

6. **WALL MOVEMENT.** For the effect on coefficients of horizontal earth pressures, see Figure 10-7.

a. Tilting Retaining Wall. For cohesionless backfill, reduction of pressures to active values requires wall rotation of only a few thousandths of a radian. However, an increase to full passive pressures requires rotation of the order of 0.05 radians. A safety factor must be applied to ultimate passive resistance to allow for incomplete mobilization of shear strength.

b. Braced Flexible Sheeting. Sheeting of narrow cuts rigidly braced at the top undergoes insufficient movement to produce fully active conditions. Horizontal pressures are distributed in a trapezoidal diagram, see Figure 10-7. The resultant force is 26 percent greater than theoretical active force in dense sands and 50 percent greater in loose sands. For clays, the intensity and distribution of horizontal pressures depend on the stability number $N_o = \gamma H/c$. See Figure 10-7.

c. Restrained Walls. Where a rigid wall is prevented from moving, horizontal pressures approach values for at-rest condition. The coefficient of earth pressures at rest K_R equals $1 - \sin \Phi$. K_R equals 0.5 for coarse grained material with Φ of 30 degrees; 0.75 for plastic clay with Φ of 15 degrees. In cohesionless soils, full at-rest pressures will occur only with the most rigidly supported walls. In highly plastic clays, pressures approaching at-rest condition may develop unless wall movement can continue with time.

7. **EFFECT OF CONSTRUCTION PROCEDURES.** For braced or anchored flexible bulkheads, pressures acting during construction, before completion of bracing or anchorage, may be critical and must be considered in design. See Section 4 for precautions to be taken in the construction of flexible bulkheads.

a. Compacted Fills. Compaction of backfill in a confined wedge behind the wall tends to increase horizontal pressures.

(1) Coarse Grained Soils. Compaction of coarse grained soils behind a rigid, unyielding wall produces horizontal pressures equal to or slightly exceeding at-rest values. For compacted fill behind flexible bulkheads or walls capable of tilting, pressures may be as low as ordinary active values.

(2) Fine Grained Soils. Pressures of fine grained soils are less influenced by compaction and are controlled by wall yield.

b. 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 in lifts thin enough to permit runoff of wash water without building up a full height of hydrostatic pressures.

8. **PRESSURES ON BINS.** See Ketchum, *The Design of Walls, Bins and Grain Elevators* (Bibliography) for methods of analyzing pressures of granular materials on bins and bunkers where active conditions are not reached because of constraint on the material.

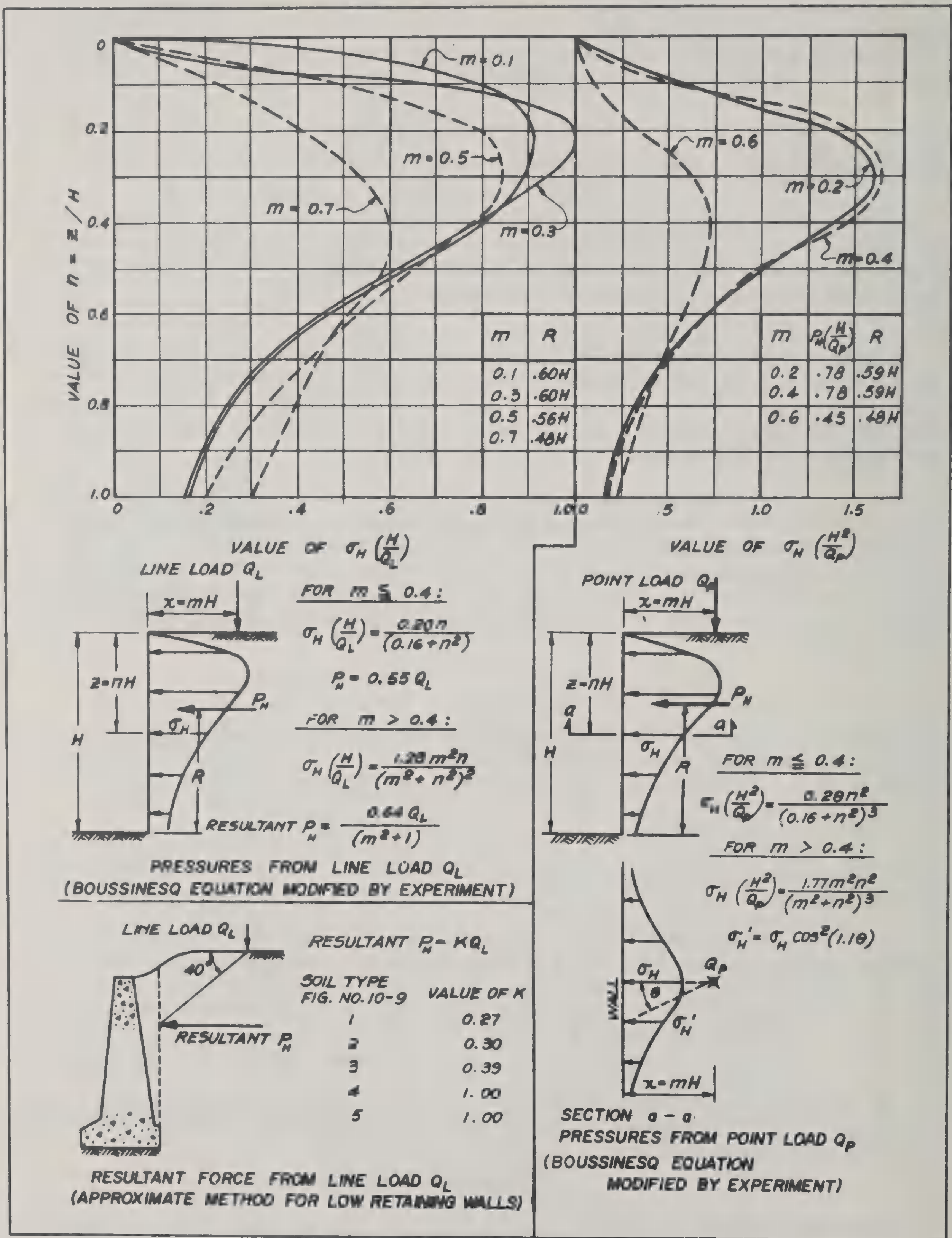
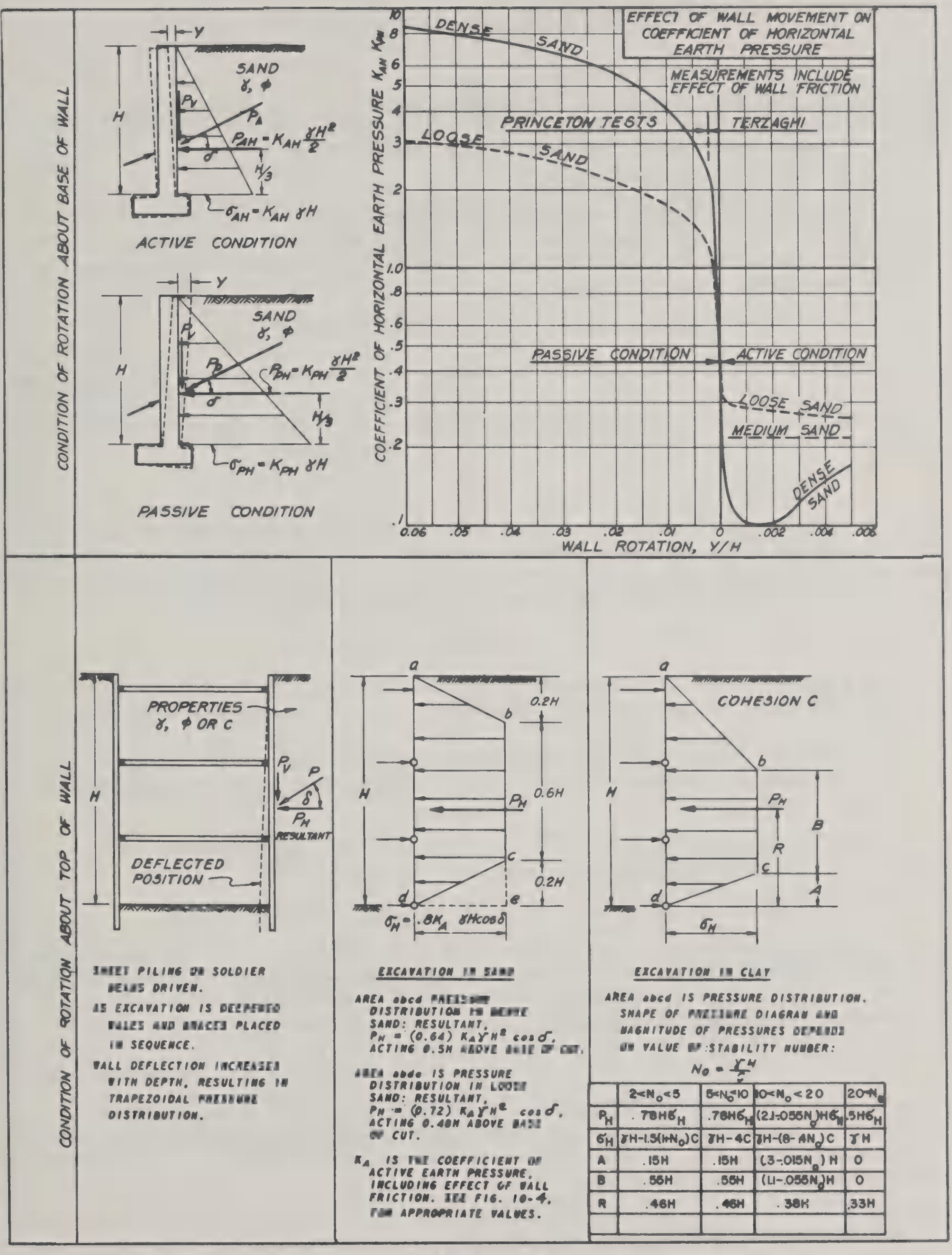


FIGURE 10-6
Horizontal Pressures on Wall From Surcharge



	$2<N_o<5$	$5<N_o<10$	$10<N_o<20$	$20<N_o$
P_H	$.78H\sigma_H$	$.78H\sigma_H$	$(2.1-.055N_o)H\sigma_H$	$.5H\sigma_H$
σ_H	$\gamma H-1.5(1+N_o)C$	$\gamma H-4C$	$\gamma H-(8-.4N_o)C$	γH
A	.15H	.15H	$(.3-.015N_o)H$	0
B	.55H	.55H	$(1.1-.055N_o)H$	0
R	.46H	.46H	.38H	.33H

FIGURE 10-7
Effect of Wall Movement on Wall Pressures

Section 3. RIGID RETAINING WALLS

1. **GENERAL CRITERIA.** Requirements for resistance against overturning and sliding of four principal wall types are given in Figure 10-8. Evaluate overall stability against deep foundation failure. (See Chapter 7.) Determine allowable bearing pressures on the base of the wall (see Chapter 11).

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, provide pile foundation or lower base of wall and consider passive resistance below frost depth.

b. Settlement and Overturning. For walls on relatively incompressible foundations apply overturning criteria of Figure 10-8. If foundation is compressible, compute settlement by methods of Chapter 6 and estimate tilt of rigid wall from the settlement. If the consequent tilt will exceed several hundredths of a radian, proportion the wall to keep the resultant force at the midpoint of base.

c. Drainage. Generally a more economical design is obtained by providing positive drainage of backfill. See Chapter 8 for drainage design. As a minimum, provide weep holes with pockets of coarse-grained material at the back of the wall. Prevent infiltration with paving or surface layer of compacted impervious soil, plus gutter for collecting runoff. See Chapter 16 for methods of minimizing frost heave forces on the wall.

2. **HIGH WALLS.** For high or expensive retaining walls, exploration and testing to define soil conditions is justified and active pressures are computed by methods in Figure 10-1. For special conditions, proceed as follows:

(1) If backfill is subject to heavy traffic vibrations, reduce tan Φ and tan δ by 20 percent.

(2) If a wall will settle more than backfill, assume wall friction to act upward against the wall.

3. **LOW WALLS.** For walls less than about 20 ft in height, detailed testing to determine soil properties and elaborate pressure computations are not justified.

a. Equivalent Fluid Pressures. Use equivalent fluid pressures of Figure 10-9 for straight slope backfill and of Figure 10-10 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 10-6. If a wall rests on a compressible foundation and moves downward with respect to the backfill, increase pressures for soil types 1, 2, 3, and 5 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 with a surface layer of impervious soil.

4. **EARTH FILLED CRIB WALLS.** See Figure 10-11 for types and design criteria. For stability against external forces, a crib wall is equivalent to gravity retaining wall. For design of structural elements against applied pressures, see Seelye, *Foundations, Design and Practice.*

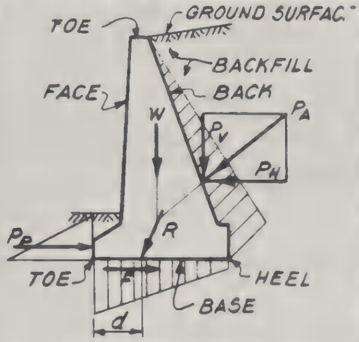
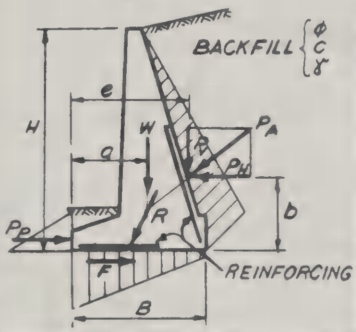
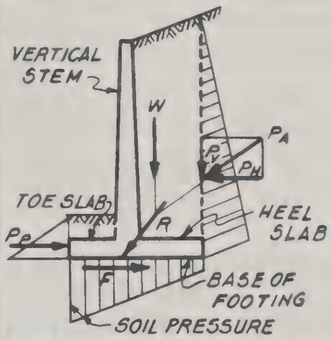
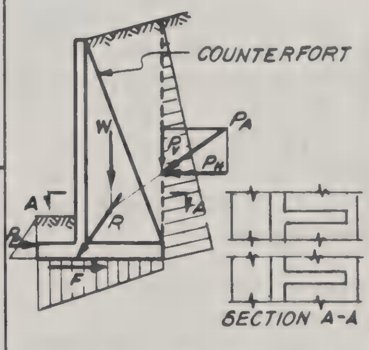
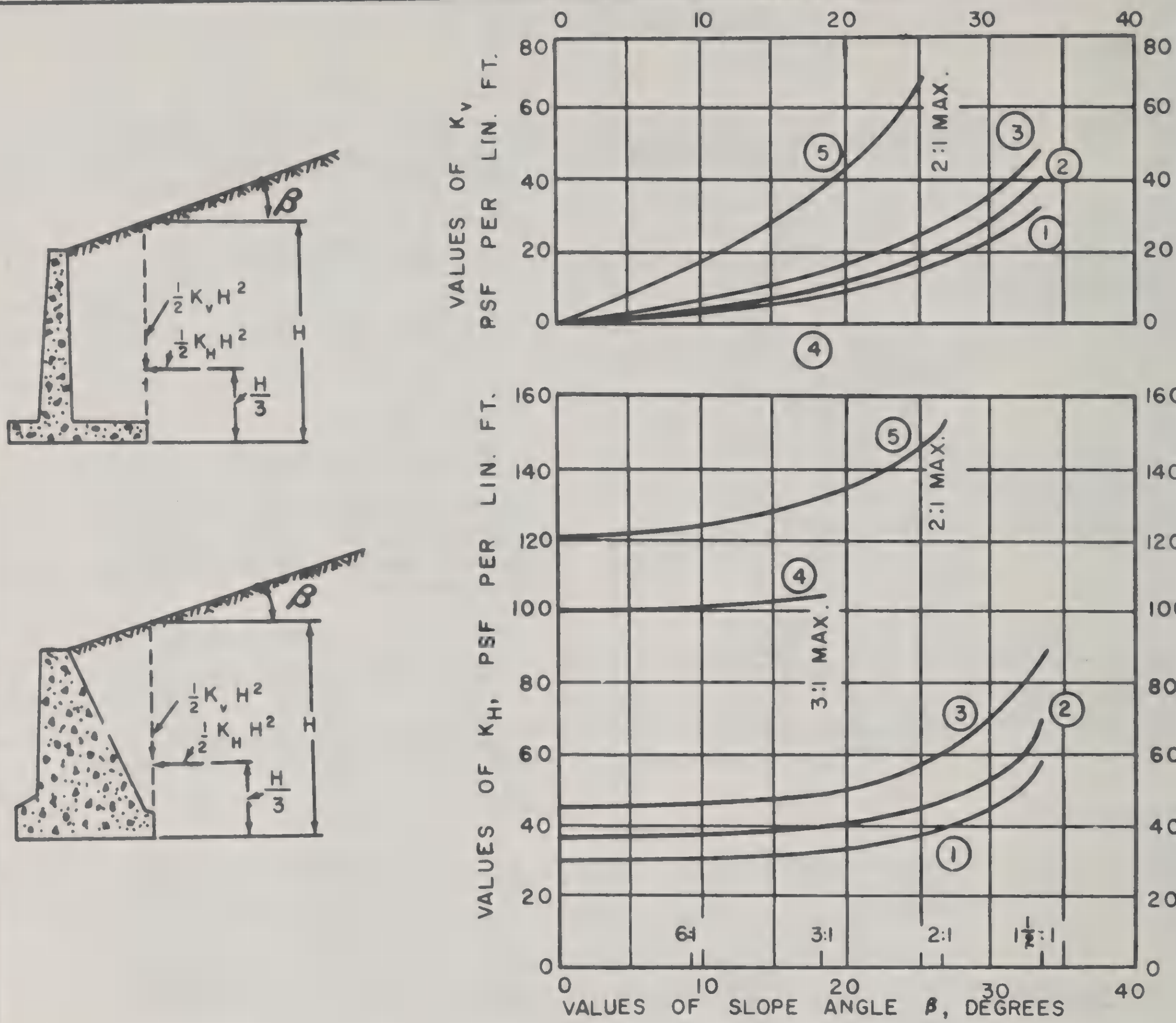
Type of wall	Load diagram	Design factors
GRAVITY		LOCATION OF RESULTANT Moments about toe: $d = \dfrac{Wa + P_V e - P_H b}{W + P_V}$ Assuming $P_p = 0$ OVERTURNING Moments about toe: $F_S = \dfrac{Wa}{P_H b - P_V e} \geq 1.5$
SEMI-GRAVITY		Ignore overturning if R is within middle third (soil), middle half (rock). Check R at different horizontal planes for gravity walls. RESISTANCE AGAINST SLIDING $F_S = \dfrac{(W + P_V) \tan \delta + C_a B}{P_H} \geq 1.5$ $F_S = \dfrac{(W + P_V) \tan \delta + C_a B + P_P}{P_H} \geq 2.0$ $F = (W + P_V) \tan \delta + C_a B$
CANTILEVER		For coefficients of friction between base and soil see Table 10-1. C_a = adhesion between soil and base $\tan \delta$ = friction factor between soil and base W = Includes weight of wall and soil in front for gravity and semi-gravity walls. Includes weight of wall and soil above footing, for cantilever and counterfort walls.
COUNTERFORT		CONTACT PRESSURE ON FOUNDATION For allowable bearing pressure for inclined load on strip foundation, see Ch. 11. For analysis of pile loads beneath strip foundation, see Ch. 13. OVERALL STABILITY For analysis of overall stability, see Ch. 7.

FIGURE 10-8
Design Criteria for Retaining Walls



CIRCLED NUMBERS INDICATE THE FOLLOWING SOIL TYPES:

1. CLEAN SAND AND GRAVEL: GW, GP, SW, SP.
2. DIRTY SAND AND GRAVEL OF RESTRICTED PERMEABILITY: GM, GM-GP, SM, SM-SP.
3. STIFF RESIDUAL SILTS AND CLAYS, SILTY FINE SANDS, CLAYEY SANDS AND GRAVELS: CL, ML, CH, MH, SM, SC, GC.
4. VERY SOFT TO SOFT CLAY, SILTY CLAY, ORGANIC SILT AND CLAY: CL; ML; OL; CH; MH; OH:
5. MEDIUM TO STIFF CLAY DEPOSITED IN CHUNKS AND PROTECTED FROM INFILTRATION: CL, CH.

FOR TYPE 5 MATERIAL H IS REDUCED BY 4 FT, RESULTANT ACTS AT A HEIGHT OF (H-4)/3 ABOVE BASE.

FIGURE 10-9
Design Loads for Low Retaining Walls (Straight Slope Backfill)

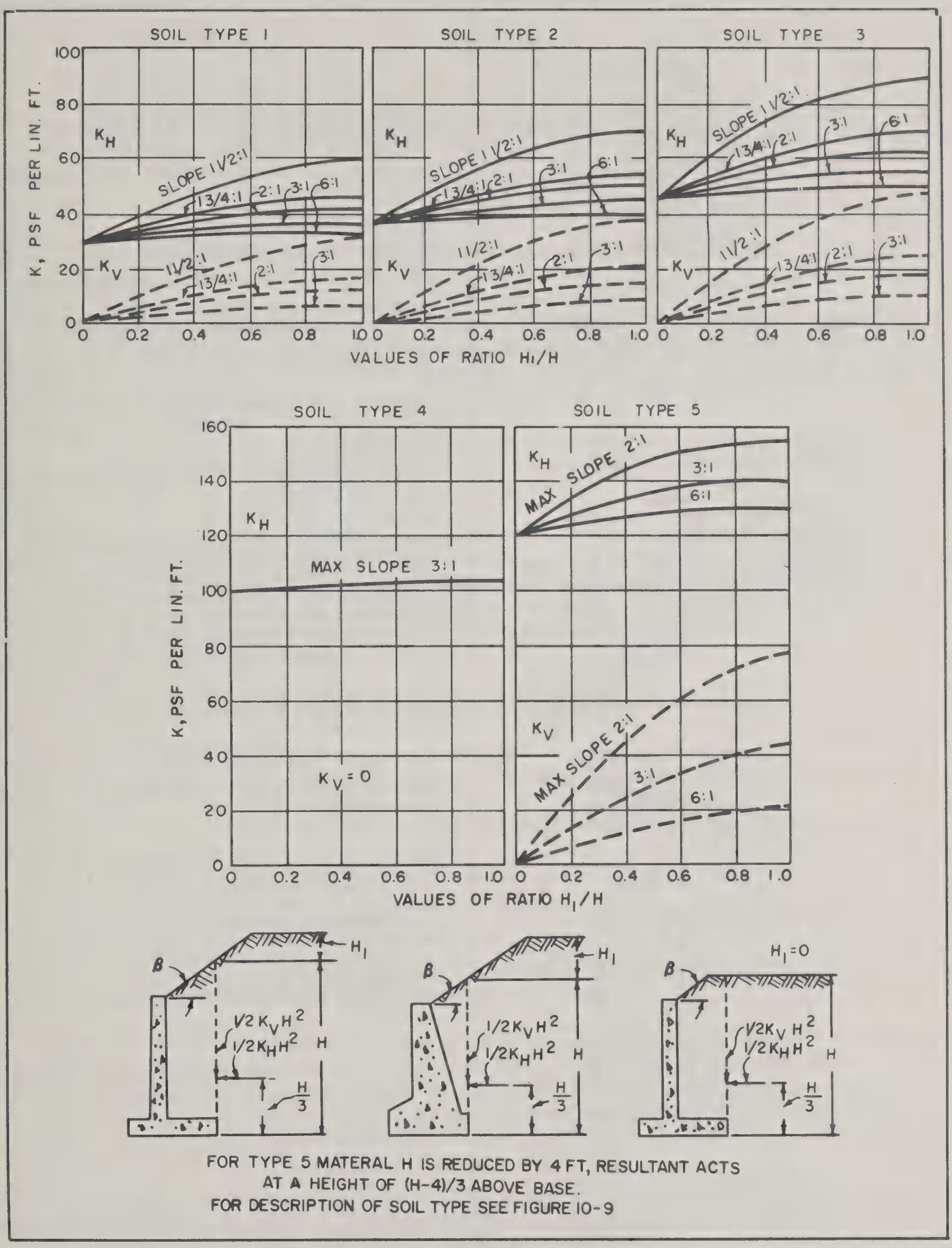
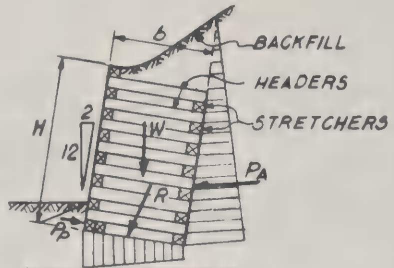
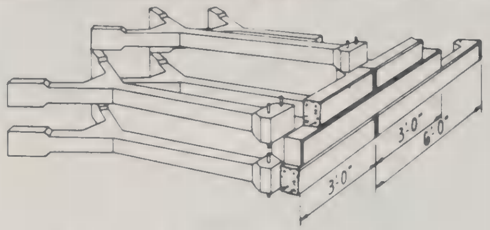


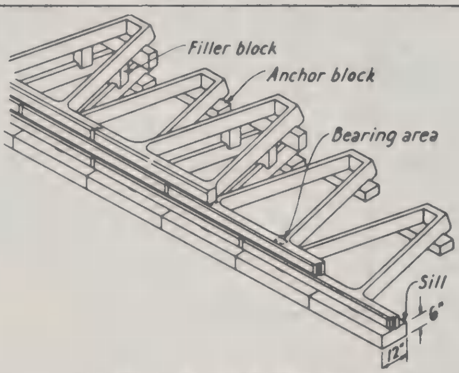
FIGURE 10-10
Design Loads for Low Retaining Walls (Broken Slope Backfill)



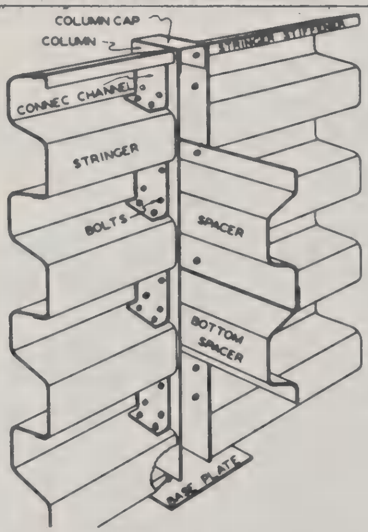
TYPICAL SECTION



FISH TAIL TYPE ASSEMBLY



CLOSE FACE ASSEMBLY



CORNER OF BIN ASSEMBLY

CRIB RETAINING WALLS

TYPES - Common types of cribs shown on accompanying diagrams.

CRIBBING MATERIALS - Timber, concrete, and metal.

FILL - Crushed stone, other coarse granular material, including rock less than 12-in. size.

DESIGN - Design criteria for gravity walls apply. Wall section resisting overturning is taken as a rectangle of dimension (H x b).

Weight of crib is equal to that of material within (H x b), including weight of crib members.
Low walls (4 ft high and under) may be made with a plumb face. Higher walls are battered on the face at least 2 in. per foot. For high walls (12 ft high and over) the batter is increased or supplemental cribs added at the back.
In open face cribs, the space between stretchers should not exceed 8 in. so as to properly retain the fill.
Expansion joints for concrete and metal cribbing are spaced no more than 90 ft.

FILLING - The wall should not be laid up higher than 3 ft above the level of the fill within the crib.

DRAINAGE & FROST ACTION - No special requirements, wall should be made free draining.

BIN TYPE RETAINING WALL - Composed of metal bins or cells joined to special columnar units at the corners. The design requirements are the same as for crib walls except that suitable drainage behind the wall is needed. Internal stresses are investigated in accordance with criteria for cellular walls.

FIGURE 10-11
Design Criteria for Crib and Bin Walls

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 10-12 for design procedures for three common penetration conditions.

a. Wall Pressures. Compute active and passive pressures by Figures 10-1 and 10-2. Determine required depth of penetration of sheeting and anchor pull from these pressures.

b. Wall Moments. 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 wall flexibility number ρ. See Figure 10-13. 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 well pressures. Where practicable, provide weep holes or special drainage at a level above mean water to limit differential water pressures.

d. Anchorage System. A tieback may be carried to a buried deadman anchorage or to pile anchorage. See Figure 10-14 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 and provide vertical support to eliminate sag. Where backfill will settle significantly or unevenly, enclose tie rod in a rigid tube to avoid loading by overburden.

e. Example of Computation. See Figure 10-15 for example of analysis of anchored bulkhead.

f. Construction Precautions. Precautions during construction are as follows:

(1) Removal of soft material, when contemplated in the design, should precede the driving of sheet piles.

(2) If existing material behind the sheeting is to be left undredged in place, dredge in front of bulkhead as necessary after completion of the wall.

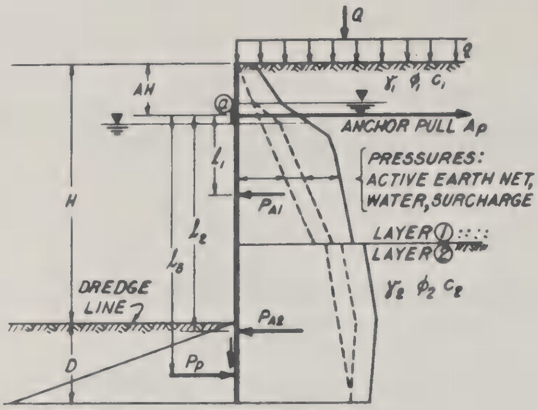
(3) If filling is to be accomplished on both active and passive sides, bring up fill evenly until passive zone is complete.

(4) Deposit backfill by working away from the wall rather than toward it to avoid trapping soft material adjacent to sheeting.

(5) Before anchorage is placed, sheeting is loaded as a cantilever wall and should be checked by procedure given in Abbett, *American Civil Engineering Practice*, Vol. II.

g. Construction Materials. Obviously, a coarse and dense backfill will impose minimum active pressures on sheeting; avoid using plastic, fine grained soils as fill behind walls. If overall economy results from obtaining and using select granular material, position the select backfill to form a plug across the potential failure surface of the active wedge. A limited volume of granular backfill placed within the limits of the active wedge produces an insignificant reduction of active pressures.

2. BRACED SHEET PILE WALLS. To restrain foundation or trench excavations, frequently braced sheet pile walls are utilized, supported by long raking braces or relatively short cross braces between trench walls. Movements of such walls range between two extremes, which control magnitude and distribution of wall pressures. Design procedures for the two limiting conditions are shown in Figure 10-16.

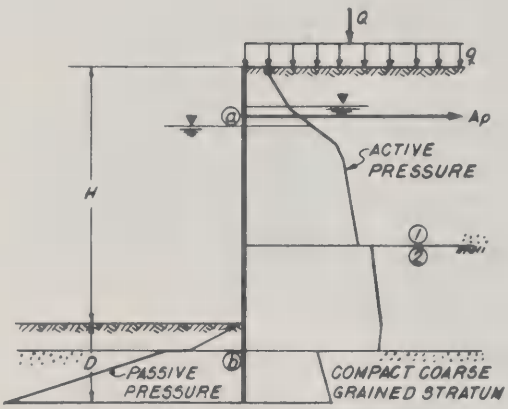


FREE EARTH SUPPORT
GENERAL CASE

1. COMPUTE PRESSURES BY METHODS OF FIGS. 10-1 TO 10-6. PASSIVE PRESSURES FOR CLEAN COARSE GRAIN SOILS INCLUDE WALL FRICTION (δ), TABLE 10-1. 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:

   $$P_{A1} l_1 + P_{A2} l_2 = \frac{P_p}{F_S} l_3$$

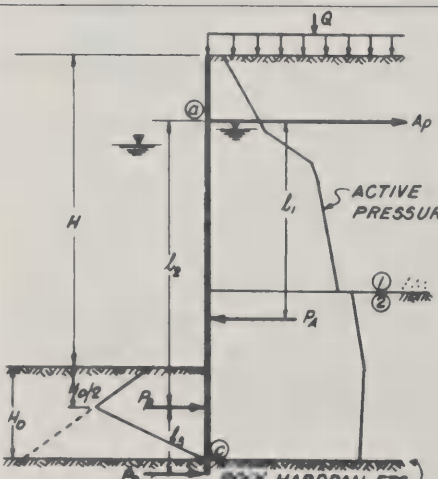
   F_S = 2 TO 3 FOR COARSE GRAINED SOILS
   F_S = 1.5 TO 2 FOR FINE GRAINED SOILS
3. ANCHOR PULL: $A_p = [P_{A1} + P_{A2} - P_p/F_S]\, d$
   d = ANCHOR SPACING.
4. MAXIMUM BENDING MOMENT ($M_{MAX.}$) IN SHEETING COMPUTED BY THE FREE EARTH SUPPORT METHOD AND APPLYING P_{A1}, P_{A2}, P_p/F_S AND A_p. FOR SHEETING IN SAND APPLY MOMENT REDUCTION FOR FLEXIBILITY OF FIG. 10-13.
5. INCREASE 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 FOR FREE EARTH SUPPORT.

4. COMPUTE MAXIMUM BENDING MOMENT (M_{MAX}) IN SHEETING BY FREE EARTH SUPPORT METHOD APPLYING P_A, P_p/F_S AND A_p.
5. COMPUTE ρ ACCORDING TO FIG. 10-13. 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 FIG. 10-13.
6. INCREASE PENETRATION COMPUTED (D) BY 20% TO ALLOW FOR DREDGING, SCOUR, ETC.



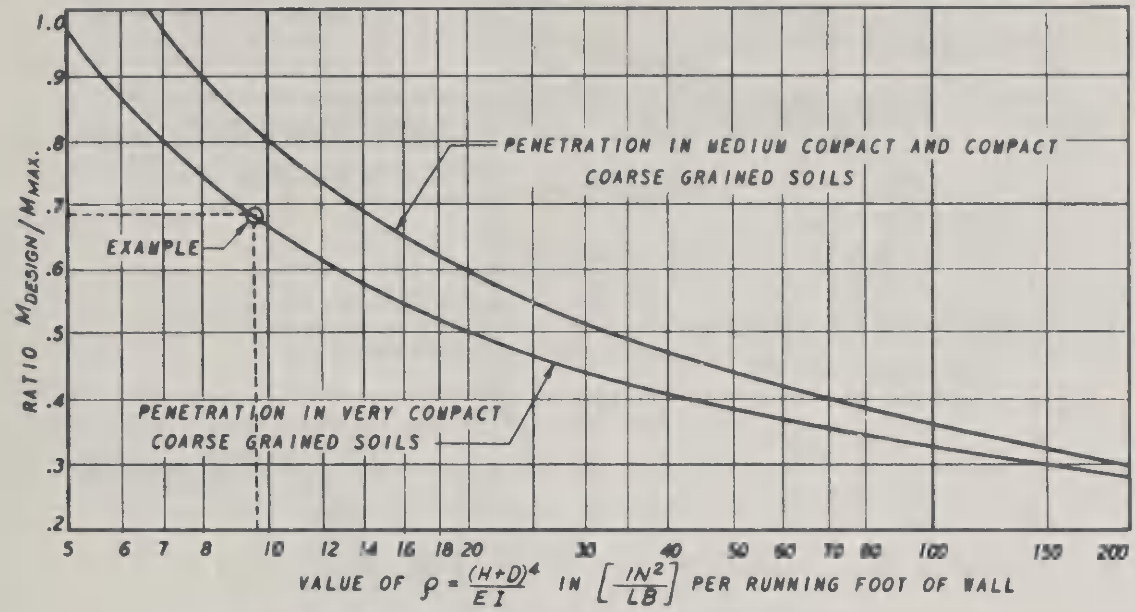
PENETRATION TO TOP OF HARD
UNYIELDING STRATUM

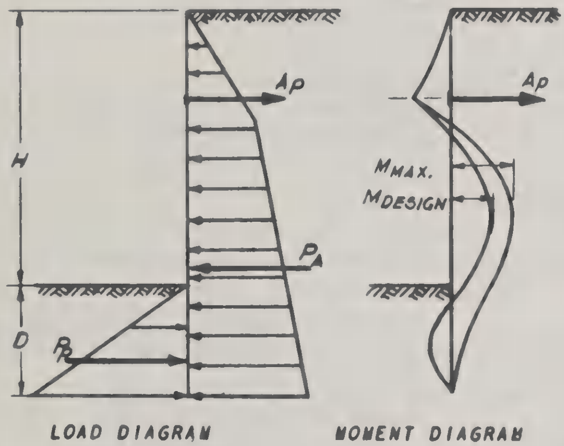
1. COMPUTE PRESSURES AS FOR FREE EARTH SUPPORT, EXCEPT THAT PASSIVE PRESSURE DECREASES TO ZERO AT TOP OF HARD STRATUM.
2. PENETRATION IN HARD STRATUM: TAKE MOMENTS ABOUT POINT ⓐ AND SOLVE FOR P_B:

   $$P_A l_1 - \frac{P_p}{F_S} l_2 = \frac{P_B}{F_S}(l_2 + l_3)$$

   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 10-12
Design Criteria for Anchored Bulkheads





EXAMPLE: 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.

LEGEND

M_{MAX} = MAXIMUM POSITIVE MOMENT IN SHEETING COMPUTED BY METHODS OF FIG. 10-12.

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.

NOTES

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 PERMITED 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 10-13
Reduction in Bending Moments in Anchored Bulkhead From Wall Flexibility

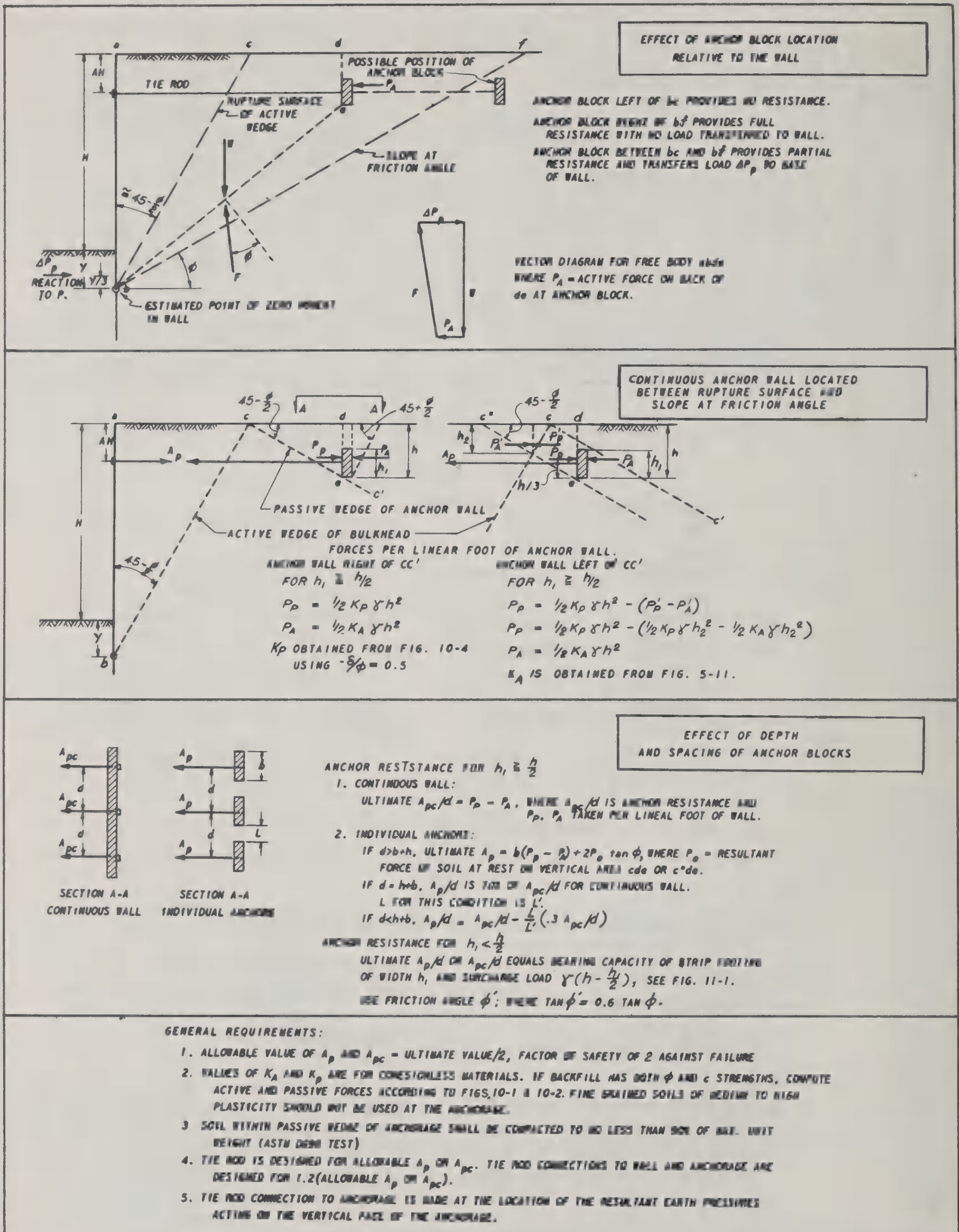
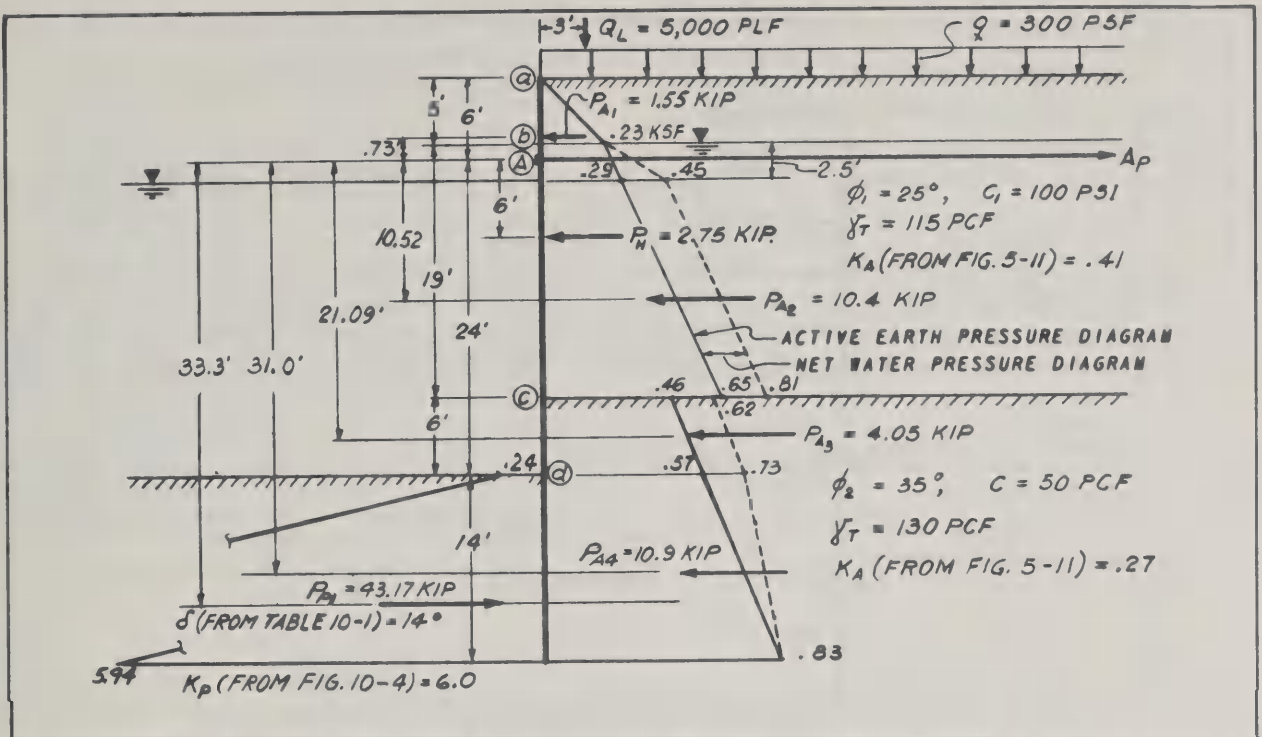


FIGURE 10-14
Design Criteria for Deadman Anchorage



ACTIVE EARTH PRESSURE (SEE FIG. 10-1) 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 FIG. 10-6).

$m = \frac{X}{H} = \frac{3}{30} = 0.1$

$P_H = 0.55\,QL = 0.55 \times 5 = 2.75$ 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 Ⓐ

$F_S = \frac{\Sigma \text{ MOMENTS OF PASSIVE FORCES}}{\Sigma \text{ MOMENTS OF ACTIVE FORCES}}$

$= \frac{43.17 \times 33.3}{1.55 \times 0.73 + 2.75 \times 6 + 10.4 \times 10.52 + 4.05 \times 21.09 + 10.9 \times 31.0}$

$= 2.61 > 2.5$

ANCHOR PULL

$A_P = \Sigma P_A - \Sigma P_P / F_S$

$= 1.55 + 2.75 + 10.4 + 4.05 + 10.9 - \frac{43.17}{2.5} = 12.37^{\,K}$

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

$M_{MAX} = 1.55 \times 15.7 + 12.37 \times 15.1 - 2.75 \times 9.1$
$- .45 \times \frac{13.6^2}{2} - .022 \times \frac{13.6^2}{2} \times 4.52 = 86.9^{\,K}$

MOMENT REDUCTION:

ASSUME: $f_S = 27{,}000$ PSI, $E = 30{,}000{,}000$ PSI

TRY ZP32, $I = 385.7\ IN^4$, $S = 38.3\ IN^4$

ρ = (FROM FIG. 10-13) $= \frac{(H+D)^4}{EI}$

$\rho = \frac{(30 \times 12 + 14 \times 12)^4}{30{,}000{,}000 \times 385.7} = 6.7\ \frac{IN^2}{LB}$

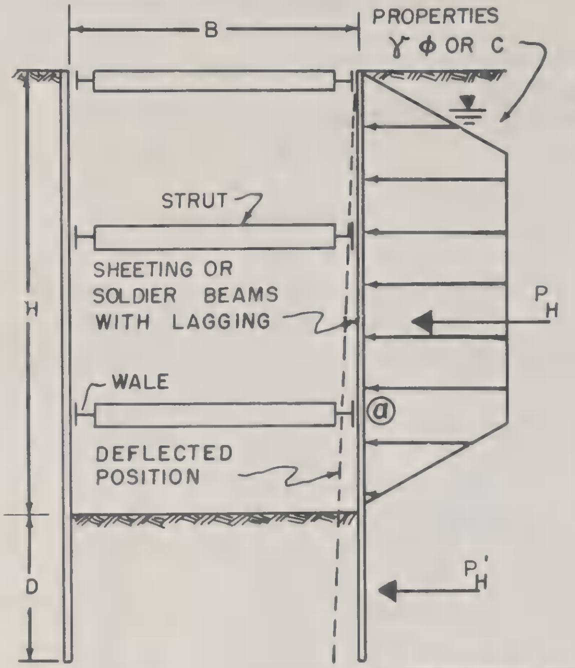
$\frac{M_{DESIGN}}{M_{MAX}} = .83$; $M_{DESIGN} = .83 \times 86.9 = 72.1^{\,K}$

$f_S = \frac{M}{S} = \frac{72.1 \times 1000 \times 12}{38.3} = 22{,}600 \text{ PSI} < 27{,}000$

TRY A SMALLER SECTION

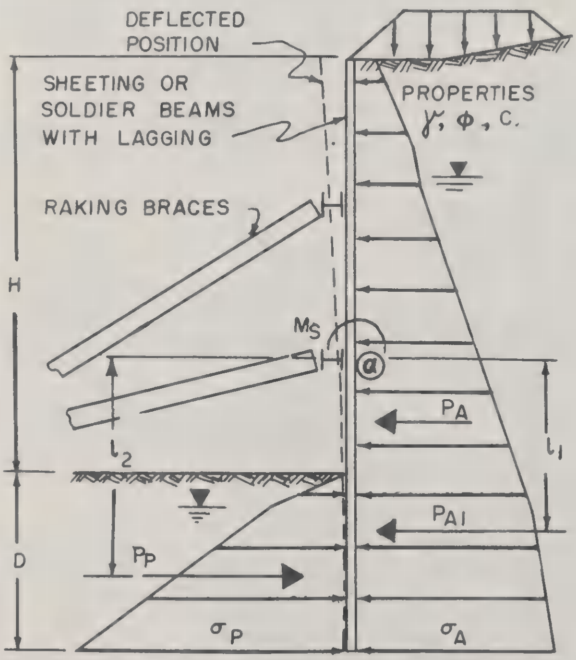
FIGURE 10-15
Example of Analysis of Anchored Bulkhead

Metz Reference Room
University of Illinois
B106 NCEL
208 N. Romine Street
Urbana, Illinois 61801



1. COMPUTE PRESSURES ON WALL ABOVE BASE OF CUT BY METHODS OF LOWER PANEL OF FIG.10-7. FOR WATER AT BACKFILL SURFACE 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 FIG. 10-17. 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 FIG. 10-17 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 × (SIMPLE SPAN MOMENTS), EXCEPT FOR UPPER SPAN WHERE MOMENT = 1.0 × (SIMPLE SPAN MOMENT). MOMENTS IN SHEETING AT POINT @ IS COMPUTED FOR CANTILEVER SPAN BELOW @, INCLUDING UNBALANCED FORCE P'_H.
4. REACTION AT BRACES COMPUTED ASSUMING SIMPLE SPAN BETWEEN BRACES.

FLEXIBLE WALL OF NARROW CUT



P_A = RESULTANT ACTIVE

P_{A1} = RESULTANT ACTIVE BELOW POINT @

1. COMPUTE ACTIVE AND PASSIVE PRESSURES BY METHODS OF FIGS. 10-1 TO 10-6. PASSIVE PRESSURES FOR CLEAN COARSE GRAINED SOILS INCLUDE WALL FRICTION (δ), TABLE 10-1. 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 USUALLY OBTAIN 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 COMPUTED 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 @, ASSUMING FIXITY AT POINT @: $P_{A1} l_1 - \frac{P_P}{F_S} l_2 - M_S = 0$

   M_S = ALLOWABLE MOMENT IN SHEETING
5. CHECK POSITIVE MOMENTS IN SPAN BELOW POINT @ FOR THIS FINAL LOADING CONDITION.

FLEXIBLE WALL WITH RAKING BRACES

FIGURE 10-16
Design Criteria for 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. Compute pressures by methods of Figures 10-1 and 10-2.

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. Compute pressures for trapezoidal distribution of Figure 10-7.

c. Stability of Base of Excavation. Where bracing restricts horizontal movement of surrounding soil, it tends to move on a vertical plane behind the wall. Analyze stability of base of cut according to Figure 10-17. For cuts in cohesionless soils, there is no base stability problem unless seepage pressures have greatly reduced passive resistance at subgrade. For cuts in cohesive soils, stability is a function of width of cut and depth to hard stratum.

d. Example of Computation. See Figure 10-18 for example of analysis of braced wall of narrow cut.

Section 5. DOUBLE-WALL COFFERDAMS

1. TYPES. Double-wall cofferdams consist of a line of circular cells connected by diaphragms, parallel straight walls or parallel walls of circular arcs with tie rods or straight connecting walls, or a succession of cloverleaf cells. For analysis, transform these types into equivalent parallel wall cofferdams of width B. See Figure 10-19, Sheet 1 of 2.

2. ANALYSIS. Stability depends on ratio of width to height, presence of an inboard berm, and type and drainage of backfill material.

a. Exterior Pressures. Ordinarily, a cell wall tilts 0.02 to 0.03 radian and active and passive pressures act on exterior faces of the sheeting.

b. Stability Requirements. A cell wall 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 10-19, Sheets 1 and 2. Criteria are given for cofferdams with and without berms, on foundation of rock or of coarse grained or fine grained soil.

(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.

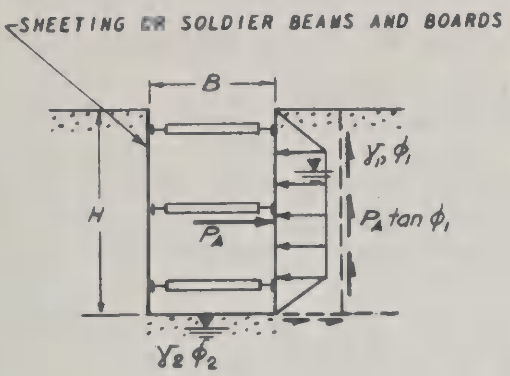
(2) Clay Base. For cofferdams on clay, penetration of outboard sheeting usually is controlled by the pullout requirement and piping is not critical.

(3) Bearing Capacity. For cofferdams on either clay or sand, check the bearing capacity at the inboard toe by methods of Chapter 11.

3. CELL FILL. Clean, coarse grained, free-draining soils are preferred for cell backfill. They may be placed hydraulically or dumped through water without compaction or special drainage.

a. Materials. If clean materials are not obtainable at reasonable cost, utilize silty sand or sand and gravel. Placement through a pipe line may eliminate most of the fines if maximum turbulence is maintained within a cell during filling. Every alternative should be studied before accepting fine grained backfill. These soils produce maximum bursting pressures and minimum cell rigidity. Their use may necessitate interior berms, increased cell width, or possibly consolidation by sand drains within the cell.

b. Drainage. Weep holes on inboard sheeting normally supply the only drainage required for clean cell fill. For dirty sand and gravel supplementary drainage by wellpoints or deep well within cells may be used to increase cell rigidity. Frequently it is economical to assume a low water level in the cell for design and to install piezometers to check that drainage provisions accomplish the necessary drawdown.



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_{γ_2} = BEARING CAPACITY FACTOR, FIG. 11-1.

IF GROUND WATER IS AT A DEPTH OF (B) OR MORE BELOW BASE OF CUT:

γ_1 AND γ_2 ARE TAKEN AS MOIST UNIT WEIGHT

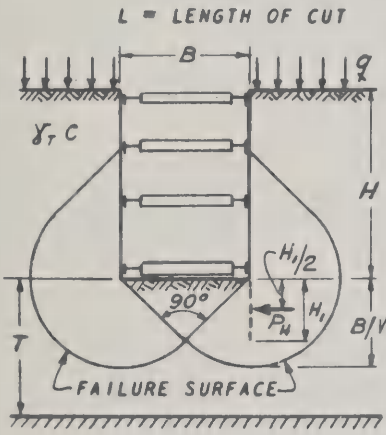
IF GROUND WATER IS STATIC AT BASE OF CUT:

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 COHESIONLESS SOIL



IF SHEETING TERMINATES AT BASE OF CUT:

SAFETY FACTOR, $F_s = \frac{N_c C}{\gamma_T H + q}$

N_c = BEARING CAPACITY FACTOR, FIG. 12-1, WHICH DEPENDS ON DIMENSIONS OF THE EXCAVATION: B, L AND H (USE H = D).

C = UNDRAINED SHEAR STRENGTH OF CLAY IN FAILURE ZONE BENEATH AND SURROUNDING BASE OF CUT.

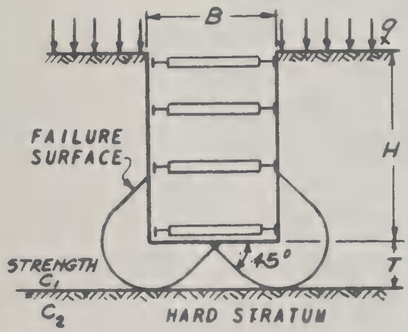
q = SURFACE SURCHARGE.

IF SAFETY FACTOR IS LESS THAN 1.5, SHEETING MUST BE CARRIED BELOW BASE OF CUT TO INSURE STABILITY, FORCE ON BURIED LENGTH:

IF $H_1 > \frac{2}{3}\frac{B}{\sqrt{2}}$, $P_H = .7(\gamma_T HB - 1.4CH - \pi CB)$

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 UNLIMITED ($T > 0.7B$)



IF SHEETING TERMINATES AT BASE OF CUT

SAFETY FACTOR:

CONTINUOUS EXCAVATION; $F_s = N_{CD}\frac{C_1}{\gamma_T H + q}$

RECTANGULAR EXCAVATION; $F_s = N_{CR}\frac{C_1}{\gamma_T H + q}$

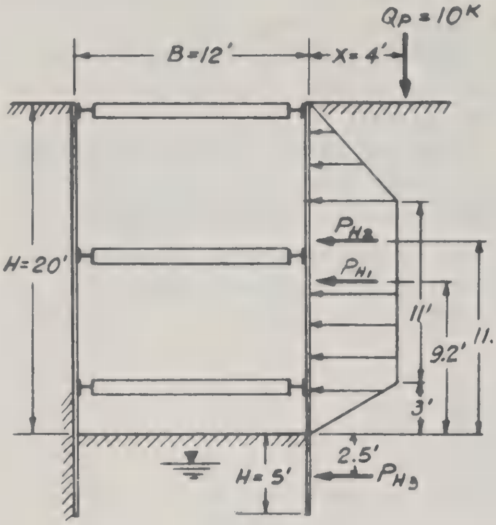
N_{CD} AND N_{CR} = BEARING CAPACITY FACTORS, FIG. 11-5, WHICH DEPEND ON DIMENSIONS OF THE EXCAVATION: B, L AND H, (USE H = D)

CUT IN CLAY, DEPTH OF CLAY LIMITED BY HARD STRATUM ($T \leq 0.7B$)

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 10-17
Stability of Base of Braced Cut



GIVEN CONDITIONS:

EXCAVATION IN SILTY CLAY.

$C = 400$ PSF $\quad \phi = 0 \quad \gamma_T = 120$ PCF

LENGTH OF EXCAVATION, $L = 80'$

DETERMINE: PRESSURES ON WALL. FORCE ON BURIED LENGTH OF SHEETING AND STABILITY OF BASE OF CUT.

PRESSURES ON WALL FROM SURROUNDING SOIL:

$$N_0 = \frac{\gamma_T H}{C} = \frac{120(20)}{400} = 6.0$$

FROM LOWER PANEL OF FIG. 10-7:

$A = .15H = 3' \qquad B = .55H = 11'$

$R = .46H = 9.2'$

$\sigma_H = \gamma_T H - 4C = (.12)(20) - 4(.4) = 0.8$ KSF

$P_{H_1} = .78H\sigma_H = .78(20)0.8 = 12.5$ KIP

PRESSURES ON WALL FROM SURCHARGE: (SEE FIG. 10-6)

$$m = \frac{X}{H} = \frac{4}{20} = .2$$

$$P_{H2} = .78\ \frac{Q_P}{H} = .78\ \frac{10}{20} = .39\ \text{KIP}$$

LOCATION OF RESULTANT;

$R = .59H = .59 \times 20 = 11.8'$

STABILITY OF BASE OF CUT: (SEE FIG. 10-17)

SAFETY FACTOR, $F_S = N_C \dfrac{C}{\gamma_T H + q}$ VALUE OF N_c IS DETERMINED BY PROCEDURES OF FIG. 12-1.

$N_C = N_{CR} = N_{CC}(1 + 0.2\frac{B}{L})$ FOR $H/B = \frac{20}{12} = 1.67$, $N_{CC} = 6.9$

$$N_C = N_{CR} = 6.9(1 + 0.2\frac{12}{80}) = 7.1$$

$F_S = 7.1\left(\dfrac{0.4}{.12 \times 20 + 0}\right) = 1.18 < 1.5$: DRIVE SHEETING BELOW BOTTOM OF EXCAVATION.

FORCE ON BURIED LENGTH OF SHEETING: (SEE FIG. 10-17)

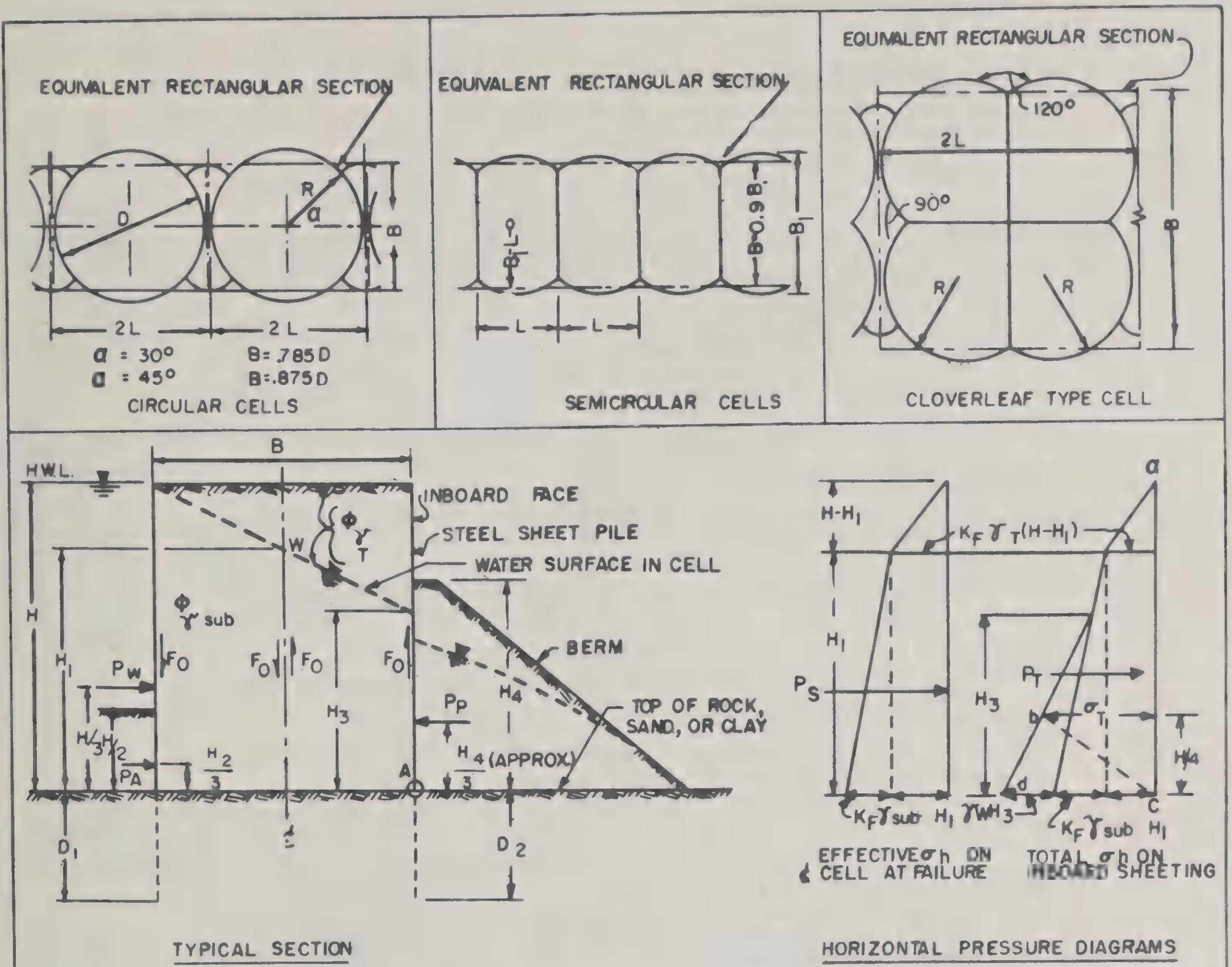
ASSUME $H_1 = 5 < \frac{2}{3}\ \frac{B}{\sqrt{2}}$. FOR $T > 0.7B$ RESULTANT FORCE P_{H_3}:

$$P_{H_3} = 1.5H_1(\gamma_T H - \frac{1.4CH}{B} - \pi C)$$

$$P_{H_3} = 1.5 \times 5\left(.12 \times 20 - \frac{1.4 \times .4 \times 20}{12} - 3.14 \times .4\right) = 1.6\ \text{KIP}$$

NOTE: ALL COMPUTATIONS ARE PER LINEAR FOOT OF WALL

FIGURE 10-18
Example of Analysis of Pressures on Flexible Wall of Narrow Cut



<u>GENERAL FACTORS</u>

SLOPE OF FREE WATER SURFACE IN CELLS DEPENDS ON PERMEABILITY OF CELL FILL.. UNLESS SPECIAL DRAINAGE IS PROVIDED AND SLOPE IS CONTROLLED, ASSUME THE FOLLOWING:
- FREE DRAINING COARSE GRAINED FILL (GW, GP, SW, SP): SLOPE 1 HORIZONTAL TO 1 VERTICAL;
- SILTY COARSE GRAINED FILL (GM, GC, SM, SC): SLOPE [illegible] TO 1;
- FINE GRAINED FILL: SLOPE [illegible] TO 1.

K_F = RATIO OF HORIZONTAL TO VERTICAL EFFECTIVE STRESS ON POTENTIAL FAILURE PLANE ON CENTERLINE OF CELL FOR $c = 0$:

$$K_F = \frac{\cos^2\phi}{2-\cos^2\phi} \quad \text{OR} \quad \sigma_h = K_F\sigma_V - K_F 2c\tan\phi \text{ (FOR } c \text{ AND } \phi).$$

<u>COFFERDAM ON ROCK - WITHOUT BERM</u>

1. ASSUME EQUIVALENT WIDTH B, USE $B = 0.85H$ FOR FIRST TRIAL
2. EFFECTIVE WEIGHT OF CELL FILL, $W = B\gamma_T(H - H_1) + BH_1\gamma_{sub}$
3. LATERAL PRESSURES PER RUNNING FOOT ON OUTBOARD FACE:
   FULL WATER PRESSURE: $P_W = \gamma_W H^2/2$
   ACTIVE EARTH PRESSURE, $P_A = K_A\gamma_{SUB}(H_2)^2/2$ (ASSUME WALL FRICTION = 0)
4. STABILITY AGAINST SLIDING ON BASE:
   $$F_S = \frac{W\tan\delta}{P_W + P_A} \geq 1.25 \text{ (TEMPORARY CONSTRUCTION)}, \geq 1.5 \text{ (PERMANENT WALL)}$$
   FOR SMOOTH ROCK: $\tan\delta = 0.5$, FOR ALL OTHERS: $\tan\delta = \tan\phi$
5. STABILITY AGAINST SHEAR FAILURE BETWEEN SHEETING AND CELL FILL:
   $$F_S = \frac{3F_0B}{P_WH + P_AH_2} \geq 1.5 \quad F_0 = P_S(\tan\delta)$$
   $\tan\delta$ FROM TABLE NO. 10-1, FOR STEEL SHEETING AGAINST CELL FILL.
6. STABILITY AGAINST SHEAR FAILURE ON CENTERLINE OF CELL:
   $$F_S = \frac{(P_S\tan\phi + P_T f)2B}{P_WH + P_AH_2} \geq 1.25 \text{ (TEMPORARY CONSTRUCTION)}, \geq 1.5 \text{ (PERMANENT WALL)}$$
   f = COEFFICIENT OF INTERLOCK FRICTION = 0.30.
   P_T = RESULTANT OF PRESSURE DIAGRAM abc.
7. INTERLOCK TENSION:
   MAXIMUM TENSION, $T_{max} = \sigma_T R$ σ_T = MAXIMUM PRESSURE OF P_T DIAGRAM abc.
8. SELECT MAXIMUM VALUE OF B WHICH SATISFIES ALL DESIGN REQUIREMENTS.

FIGURE 10-19
Design Criteria for Cellular Cofferdams

COFFERDAM ON ROCK – WITH BERM

DESIGN STEPS 1 TO 3 SAME AS COFFERDAM ON ROCK WITHOUT BERM.

4. STABILITY AGAINST SLIDING ON BASE:

$$F_s = \frac{W \tan\delta}{P_W + P_A - P_P} \geq 1.25 \text{ (TEMPORARY CONSTRUCTION)}, \geq 1.5 \text{ (PERMANENT WALL)}$$

P_P = RESULTANT PASSIVE FORCE ON BERM, INCLUDING EFFECT OF WALL FRICTION, $\tan\delta$, FROM TABLE 10-1.

P_P MUST $\leq$ (EFFECTIVE WEIGHT OF BERM) $\times \tan\delta$ OR FAILURE WILL OCCUR ON BASE OF BERM.

5. STABILITY AGAINST SHEAR FAILURE BETWEEN SHEETING AND CELL FILL:

$$F_S = \frac{3 F_O B}{P_W H + P_A H_2 - P_P H_4} \geq 1.3$$

6. STABILITY AGAINST SHEAR FAILURE ON CENTERLINE OF CELL:

$$F_s = \frac{2B(P_3 \tan\phi + (P_T - P_3) f)}{P_W H + P_A H_2 - P_P H_4} \geq 1.25 \text{ (TEMPORARY CONSTRUCTION)}, \geq 1.8 \text{ (PERMANENT WALL)}$$

P_T = RESULTANT OF PRESSURE DIAGRAM adc, NOT INCLUDING WATER PRESSURE $\gamma_W H_3$.

7. INTERLOCK TENSION SAME AS FOR COFFERDAM WITHOUT BERM.

COFFERDAM ON DEEP SAND FOUNDATION – WITHOUT BERM

DESIGN STEPS 1, 2, 3, 4 AND 7 SAME AS FOR COFFERDAM ON ROCK EXCEPT THAT P_T IS RESULTANT OF ENTIRE AREA $abdc$. STEPS 4 AND 5 DO NOT CONTROL.

8. STABILITY AGAINST BEARING CAPACITY FAILURE OF INBOARD TOE:

$$F_s = \frac{\text{ULTIMATE BEARING CAPACITY}}{\frac{W}{B}\left(1 + \frac{6e}{B}\right)} \geq 2 \qquad e = \frac{B}{2} - \frac{P_W\left(\frac{H}{3}\right) + P_A\left(\frac{H_2}{3}\right) - W\left(\frac{B}{2}\right)}{W}$$

USE ULTIMATE BEARING CAPACITY FOR CONTINUOUS FOOTING OF WIDTH B, FIG. 11-1.

PENETRATION OF SHEET PILING MAY DEPEND ON UNDERSEEPAGE CUTOFF REQUIREMENTS WHICH ARE EVALUATED WITH FLOW NET.

IN GENERAL, TO AVOID PIPING AT INBOARD TOE: $D_1 = D_2 = \frac{2}{3}H$

OR $D_1 = D_2 = H/2$ IF WATER LEVEL IS LOWERED AT LEAST $\frac{H}{6}$ BELOW INBOARD GROUND SURFACE.

PENETRATION TO AVOID PULL-OUT OF OUTBOARD SHEETING.

$$\frac{Q_U}{Q_P} \geq 1.5$$

Q_U = ULTIMATE PULL OUT CAPACITY PER LINEAR FOOT OF WALL, SEE CH13 -SEC 2.

$$Q_P = \frac{P_W H + P_A H_2}{3B(1 + B/4L)}$$

COFFERDAM ON DEEP SAND FOUNDATION – WITH BERM

DESIGN STEPS 1, 2, 3, 6 AND 7 SAME AS FOR COFFERDAM ON DEEP SAND. FOUNDATION WITHOUT BERM, EXCEPT THAT PASSIVE RESULTANT P_P IS INCLUDED IN RESISTING OVERTURNING MOMENT.

8. STABILITY AGAINST BEARING CAPACITY FAILURE IS NOT CRITICAL WITH PRESENCE OF BERM.

PENETRATION OF SHEETING REQUIRED TO AVOID PIPING IS EVALUATED WITH FLOW NET.

PENETRATION OF OUTBOARD SHEETING TO AVOID PULL-OUT SAME AS FOR COFFERDAM ON DEEP SAND WITHOUT BERM EXCEPT THAT $P_P(H_4/3)$ IS INCLUDED IN RESISTING MOMENT FOR Q_P.

COFFERDAM ON STIFF TO HARD CLAY

DESIGN STEPS 1, 2, 3, 6 AND 7 SAME AS FOR COFFERDAM ON SAND.

8. STABILITY AGAINST BEARING CAPACITY FAILURE OF INBOARD TOE SAME AS FOR COFFERDAM ON DEEP SAND EXCEPT THAT $F_s \geq 2.5$.

PENETRATION REQUIRED TO AVOID PIPING USUALLY IS NOT IMPORTANT.

PENETRATION OF OUTBOARD SHEETING TO AVOID PULL-OUT IS EVALUATED BY SAME CRITERIA AS FOR COFFERDAM ON SAND.

COFFERDAM ON SOFT TO MEDIUM STIFF CLAY

DESIGN STEPS 1, 2, 3 AND 7 SAME AS FOR COFFERDAM ON SAND.

6. STABILITY AGAINST SHEAR FAILURE ON CENTERLINE OF CELL:

$$\frac{(P_T - P_D) Rf \left(\frac{B}{L} \times \frac{L + 0.25B}{L + 0.50B}\right)}{P_W H + P_A H_2 - P_P H_4} \geq 1.25 \text{ (TEMPORARY CONSTRUCTION)}, \geq 1.5 \text{ (PERMANENT WALL)}$$

8. STABILITY AGAINST BEARING CAPACITY FAILURE OF INBOARD TOE SAME AS FOR COFFERDAM ON DEEP SAND EXCEPT THAT $F_s \geq 3$.

PENETRATION REQUIRED TO AVOID PIPING USUALLY IS NOT IMPORTANT.

PENETRATION OF OUTBOARD SHEETING TO AVOID PULL-OUT IS EVALUATED BY SAME CRITERIA AS FOR COFFERDAM ON SAND.

FIGURE 10-19 (Continued)
Design Criteria for Cellular Cofferdams

CHAPTER 11. SPREAD FOUNDATIONS

Section 1. INTRODUCTION

1. **SCOPE.** This chapter presents criteria for the design of spread footings and mats, methods of determining allowable bearing pressures, and treatment of problems in swelling subsoil. Spread foundations are of two general types; column, wall, or combined footings supporting the load of one or more structural units, and mats beneath the entire building area. This chapter also includes guidance on anchorages for tower guys.

2. **RELATED CRITERIA.** See NAVFAC DM-2 for criteria for loads applied to foundations by various structures and structural design of foundations.

3. **APPLICATIONS.** Use spread foundations at shallow depths where settlement or stability of underlying material does not threaten damage to structures. Where a suitable bearing stratum at ground surface is underlain by weaker and more compressible materials, consider the use of deep foundations or piles. See Chapters 12 and 13.

Section 2. BEARING CAPACITY

1. **LIMITATIONS.** Allowable bearing pressures on spread 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.

2. **ULTIMATE SHEAR FAILURE.** For major structures, where relatively high foundation bearing pressures yield important economy, determine ultimate bearing capacity by detailed exploration, laboratory testing, and theoretical analysis. For small or temporary structures, estimate allowable bearing pressures from performance of nearby buildings or from nominal bearing values; see Paragraph 4.

a. **Theoretical Bearing Capacity.** To analyze ultimate bearing capacity for various loading situations, see Figures 11-1 through 11-5. 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. See Figures 11-6 and 11-7 for examples of safety factor computations. To obtain allowable bearing pressures, apply a safety factor between 2 and 3 for dead load plus maximum live load, depending on the nature of the structure and the reliability with which subsoil conditions have been determined. Include in design dead load the effective weight of footing and soil directly above base of footing.

b. **Bearing Capacity Diagrams.** Utilize ultimate bearing capacity diagrams as follows:

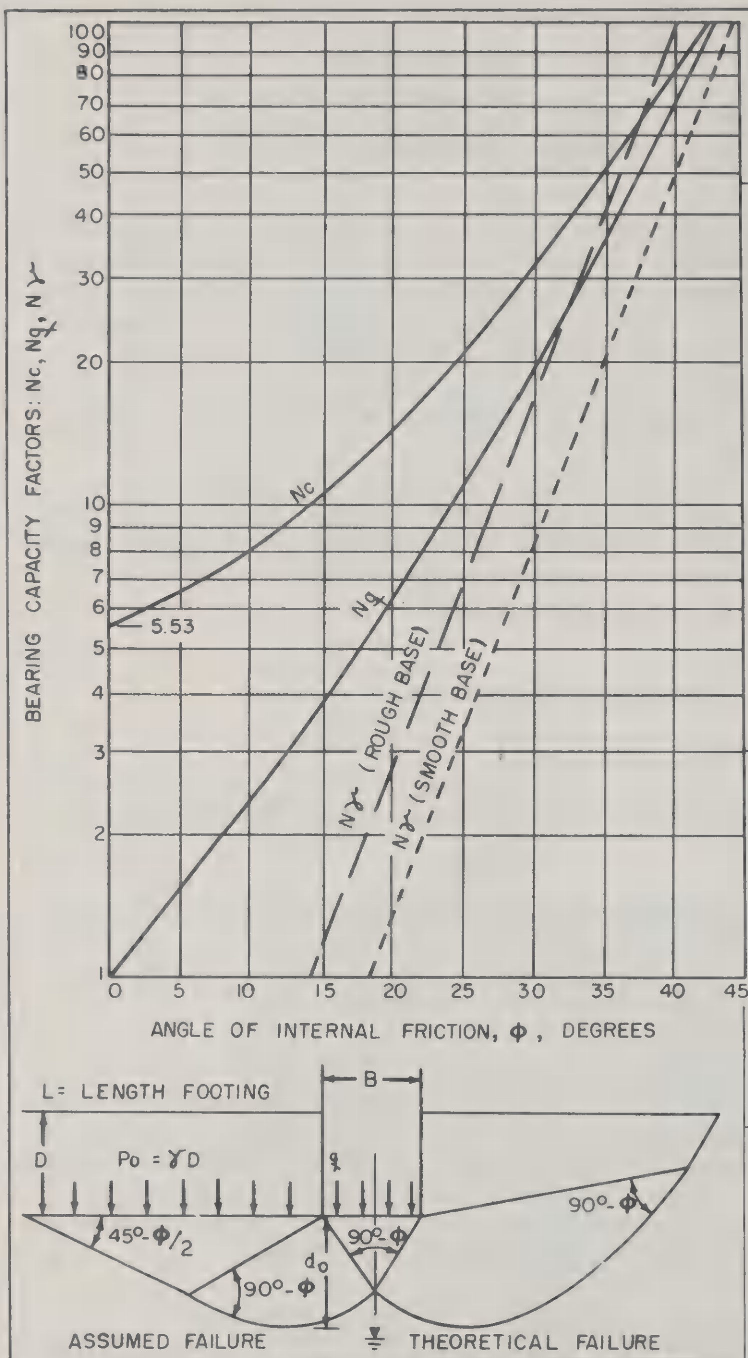
(1) See Figure 11-1 for shallow footings with concentric vertical load. Formulas shown assume ground water at a depth below base of footing equal to the narrow dimension of the footing.

(2) Use Figure 11-2 to determine ground water effect on ultimate bearing capacity. For cohesive soils, changes in ground water level do not affect ultimate bearing capacity.

(3) Use Figure 11-3 for inclined load on horizontal footing and for inclined load on inclined footing.

(4) Use Figure 11-4 for shallow footing with concentric vertical load placed on a slope or near top of slope.

(5) Use Figure 11-5 for shallow footing with concentric vertical load on layered cohesive soil.



ASSUMED CONDITIONS:

1. $D \leq B$
2. SOIL IS UNIFORM TO DEPTH $d_0 > B$
3. WATER LEVEL LOWER THAN d_0 BELOW BASE OF FOOTING.
4. VERTICAL LOAD CONCENTRIC.
5. FRICTION AND ADHESION ON VERTICAL SIDES OF FOOTING ARE NEGLECTED.
6. FOUNDATION SOIL WITH PROPERTIES c, ϕ, γ

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 SOIL

$q' = cN_c + \gamma D N_q$

$q'' = \gamma \frac{B}{2} N_\gamma$

$q_{ult} = cN_c + \gamma D N_q + \frac{\gamma B}{2} N_\gamma$

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: RADIUS = R

$q_{ult} = 1.3\, cN_c + \gamma D N_q + 0.6\gamma R N_\gamma$

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:

$q_{ult} = cN_c + \gamma D$

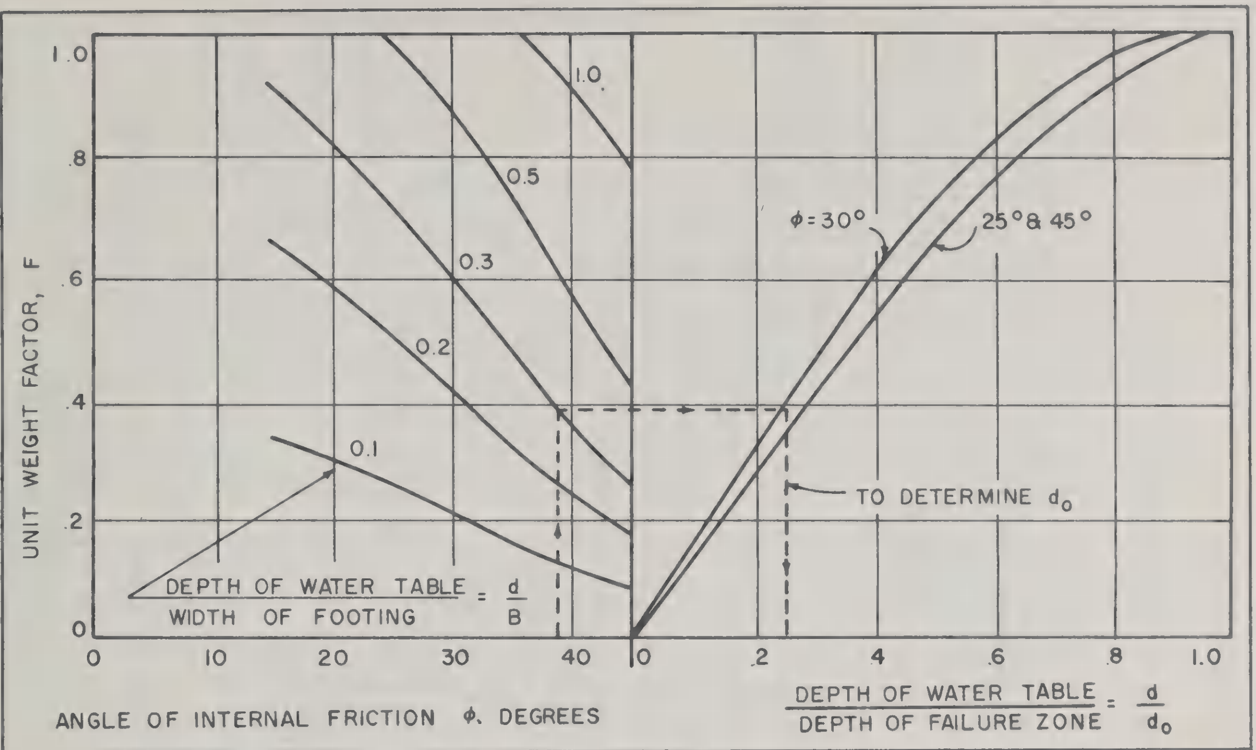
SQUARE OR RECTANGULAR FOOTING:

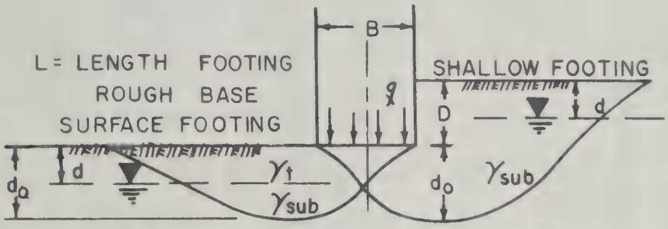
$q_{ult} = cN_c(1+.3\frac{B}{L}) + \gamma D$

CIRCULAR FOOTING:

$q_{ult} = 1.3\, cN_c + \gamma D$

FIGURE 11-1
Ultimate Bearing Capacity of Shallow Footings With Concentric Loads





ASSUMED CONDITIONS:

1. GROUND WATER LEVEL IS HORIZONTAL
2. PRESENCE OF GROUND WATER 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: $D \leqq B$

IF $d \leqq D$

$$q_{ult} = cN_c + [\gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d] N_q + 0.5 \gamma_{SUB} B N_\gamma$$

IF $D < d \leqq (D + d_o)$

$$q_{ult} = cN_c + \gamma_T D N_q + [\gamma_{SUB} + F(\gamma_T - \gamma_{SUB})] \frac{B}{2} N_\gamma$$

VALUES OF BEARING CAPACITY FACTORS N_c, N_q AND N_γ ARE SHOWN IN FIG. 11-1.

RECTANGULAR FOOTING:

SURFACE FOOTING: $D = 0$

$$q_{ult} = cN_c(1 + .3\tfrac{B}{L}) + [\gamma_{SUB} + F(\gamma - \gamma_{SUB})] 0.4 B N_\gamma$$

SHALLOW FOOTING: $D \leqq B$, IF $d \leqq D$

$$q_{ult} = cN_c(1 + .3\tfrac{B}{L}) + [\gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d] N_q + 0.4 \gamma_{SUB} B N_\gamma$$

IF $D < d \leqq (D + d_o)$

$$q_{ult} = cN_c(1 + .3\tfrac{B}{L}) + \gamma_T D N_q + [\gamma_{SUB} + F(\gamma_T - \gamma_{SUB})] 0.4 B N_\gamma$$

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: $D \leqq 2R$, IF $d \leqq D$

$$q_{ult} = 1.3 cN_c + [\gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d] N_q + 0.6 \gamma_{SUB} R N_\gamma$$

IF $D < d \leqq (D + d_o)$

$$q_{ult} = 1.3 cN_c + \gamma_T D N_q + [\gamma_{SUB} + F(\gamma_T - \gamma_{SUB})] 0.6 R N_\gamma$$

FIGURE 11-2
Ultimate Bearing Capacity With Ground Water Effect

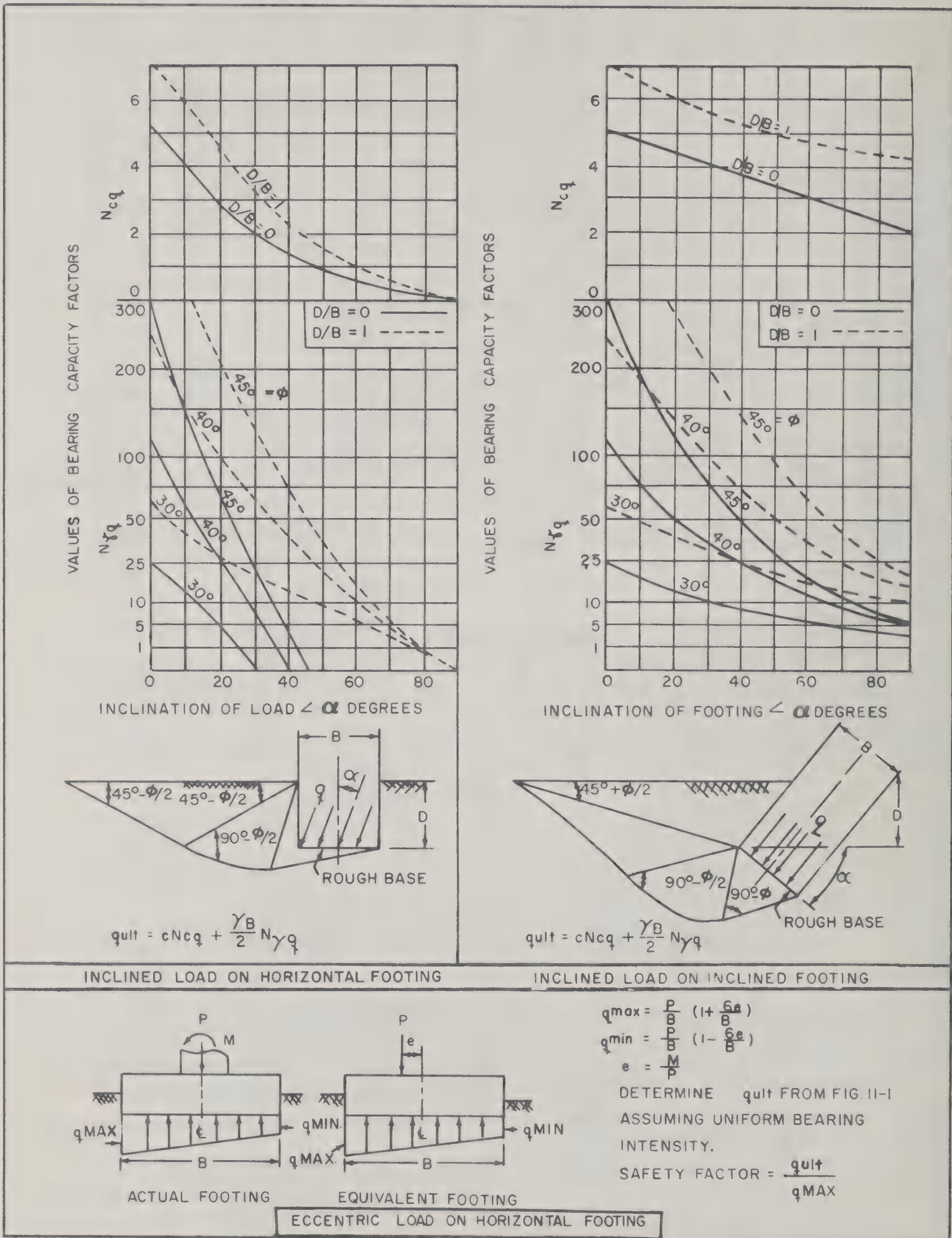
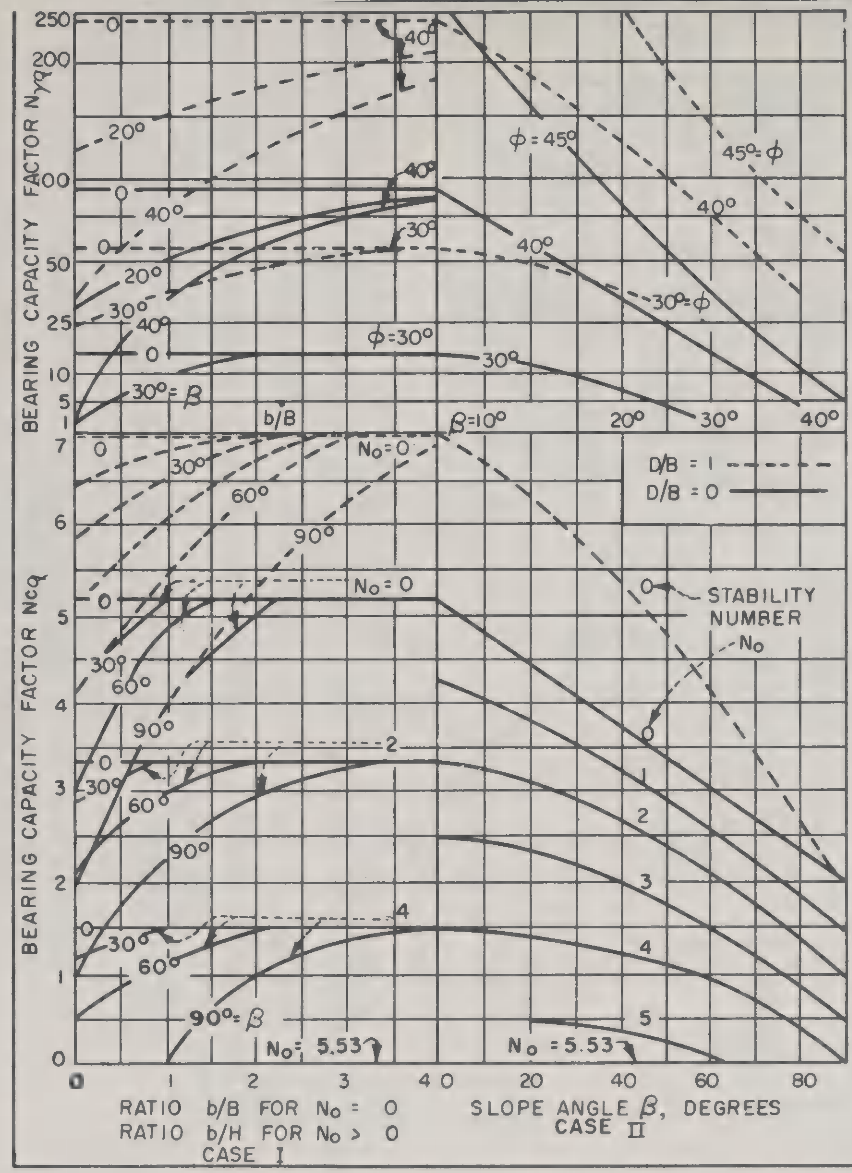
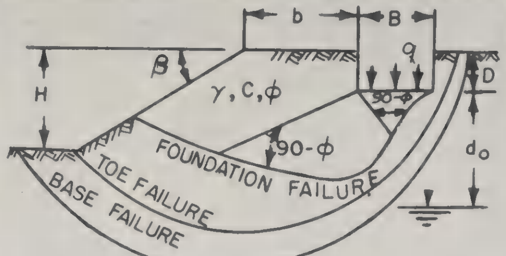


FIGURE 11-3
Ultimate Bearing Capacity of Continuous Footings With Eccentric or Inclined Loads



CASE I: CONTINUOUS FOOTING AT TOP OF SLOPE



WATER AT $d_o \geqq B$:

$$q_{ult} = cN_{cq} + \gamma_T \frac{B}{2} N_{\gamma q} \quad (1)$$

WATER AT GROUND SURFACE

$$q_{ult} = cN_{cq} + \gamma_{sub} \frac{B}{2} N_{\gamma q} \quad (2)$$

IF $B \leqq H$:

OBTAIN N_{cq} FROM DIAGRAM 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 DIAGRAM FOR CASE I WITH STABILITY NUMBER $N_o = \frac{\gamma H}{C}$. INTERPOLATE FOR VALUES $0 < D/B < 1$.

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 EQ (2)

$$\phi' = \tan^{-1}\left(\frac{\gamma_{sub}}{\gamma_T} \tan\phi\right)$$

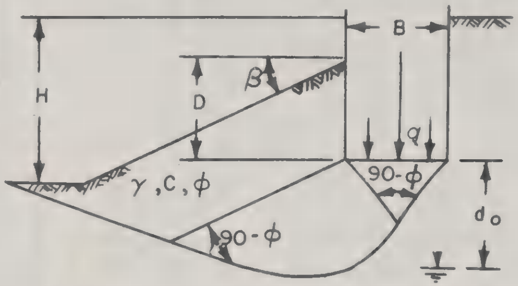
COHESIVE SOIL ($\phi = 0$)

SUBSTITUTE IN EQ (1) AND (2) D FOR $B/2$ AND $N_{\gamma q} = 1$.

RECTANGULAR, SQUARE, OR CIRCULAR FOOTING:

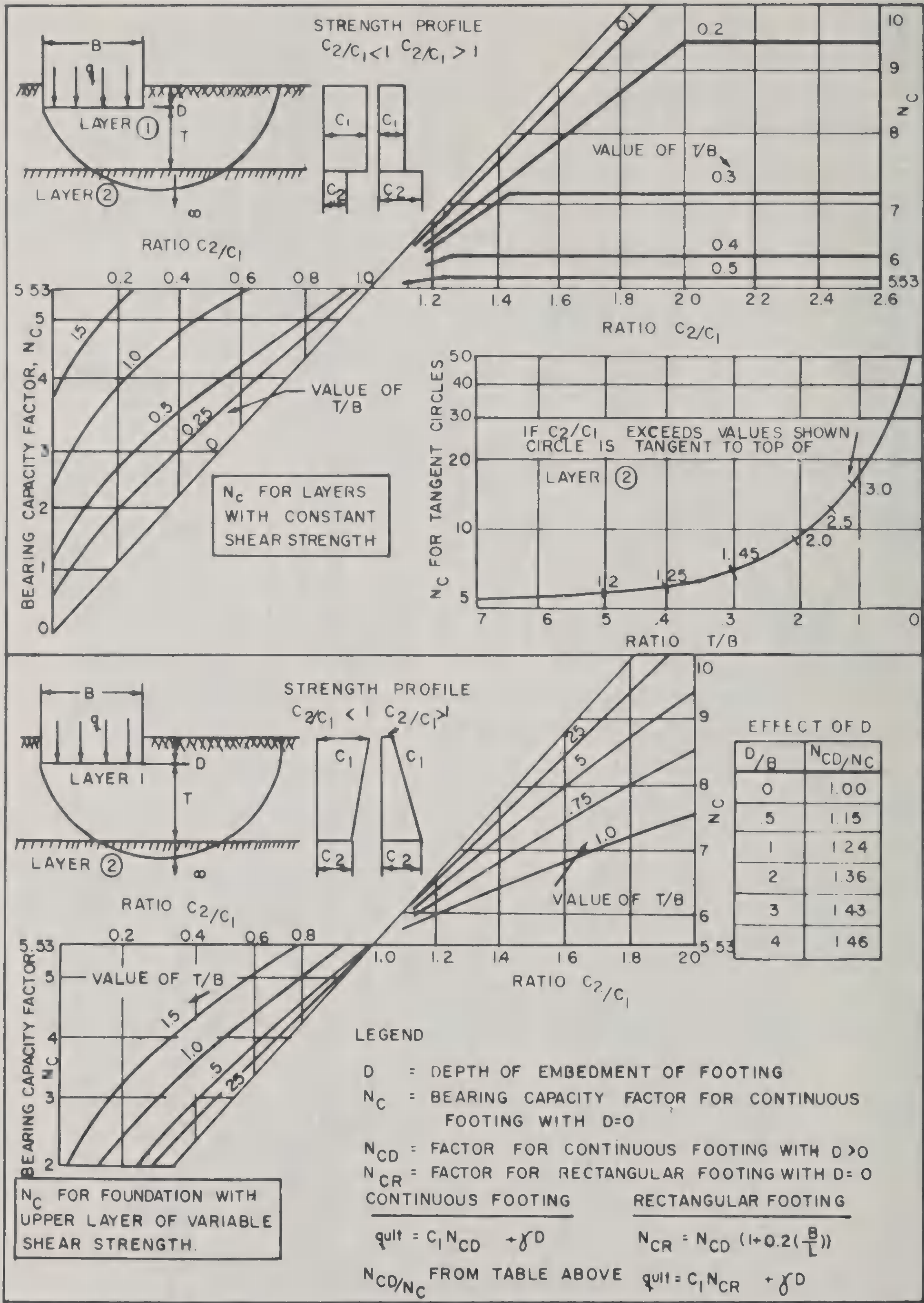
$$q_{ult} = \left[q_{ult} \text{ FOR CONTINUOUS FOOTING AS GIVEN ABOVE}\right] \times \left[\frac{q_{ult} \text{ FOR FINITE FOOTING}}{q_{ult} \text{ FOR CONTINUOUS FOOTING}}\right] \text{ FROM FIG. 11-1}$$

CASE II: CONTINUOUS FOOTING ON SLOPE



SAME CRITERIA AS FOR CASE I EXCEPT THAT N_{cq} AND $N_{\gamma q}$ ARE OBTAINED FROM DIAGRAMS FOR CASE II

FIGURE 11-4
Ultimate Bearing Capacity of Shallow Footings on Slope



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

LEGEND

D = DEPTH OF EMBEDMENT OF FOOTING

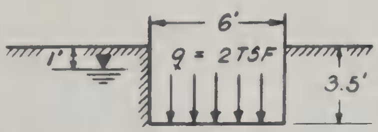
N_C = BEARING CAPACITY FACTOR FOR CONTINUOUS FOOTING WITH D=0

N_{CD} = FACTOR FOR CONTINUOUS FOOTING WITH D > 0

N_{CR} = FACTOR FOR RECTANGULAR FOOTING WITH D = 0

CONTINUOUS FOOTING	RECTANGULAR FOOTING
$q_{ult} = C_1 N_{CD} + \gamma D$	$N_{CR} = N_{CD}\,(1+0.2(\frac{B}{L}))$
N_{CD}/N_C FROM TABLE ABOVE	$q_{ult} = C_1 N_{CR} + \gamma D$

FIGURE 11-5
Ultimate Bearing Capacity on Two Layer Cohesive Soil (ϕ = 0)



$C = 2.0$ KSF $\quad \phi = 0$

$\gamma_T = 130$ PCF

$q = \begin{cases} DL = 1.0 \text{ TSF} \\ LL = 1.0 \text{ TSF} \end{cases}$

DEAD LOADS INCLUDES EFFECTIVE WEIGHT OF FOOTING WITH ASSUMED GROUND WATER LEVEL.

N_c (FROM FIG. 11-1) = 5.53

K_{v_1} (FROM FIG. 11-8) = $100 T/FT^3$

$q_{ult} = CN_C + \gamma D$ $\quad \{ \gamma D$ = EFFECTIVE SURCHARGE PRESSURE AT LEVEL OF BASE OF FOOTING.

$q_{ult} = 2(5.53) + [.13(1) + (.13 - .0625)(2.5)] = 11.4$ KSF

SAFETY FACTOR, $F_S = \frac{q_{ult}}{q} = \frac{11.4}{4} = \underline{2.85}$

IMMEDIATE SETTLEMENT $\delta_{vo} = \frac{1.05 q B C_S \sqrt{L/B}}{K_{v1}}$

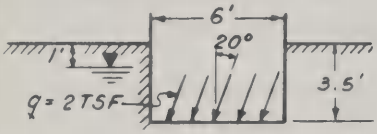
TAKE $L/B = 10$ $\quad C_S$ (FROM FIG. 11-9) = .68 RIGID CASE.

$\delta_{vo} = \frac{1.05(2)(6)(.68)\sqrt{10}}{100} = 0.27$ FT

$\frac{D}{\sqrt{BL}} = \frac{3.5}{\sqrt{6(60)}} = 0.18$ $\quad \delta_{vd}/\delta_{vo}$ (FROM FIG. 11-9) = 0.94

$\delta_{vd} = 0.94(0.27)(12) = \underline{3.1 \text{ IN.}}$

Ⓐ CONTINUOUS FOOTING WITH VERTICAL LOAD



$C = 2.0$ KSF $\quad \phi = 0 \quad q = 2$ TSF

$\gamma_T = 130$ PCF

$D/B = \frac{3.5}{6} = 0.58$

N_{cq} (FROM FIG. 11-3) = 3.8 $\quad N_{\gamma q} = 0$

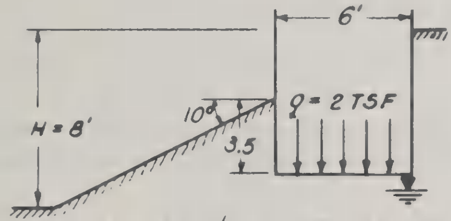
$q_{ult} = CN_{cq} + \frac{\gamma B}{2} N_{\gamma q} = 2(3.8) + 0 = 7.6$ KSF

SAFETY FACTOR, $F_S = \frac{q_{ult}}{q} = \frac{7.6}{4} = \underline{1.90}$

IMMEDIATE SETTLEMENT, NO EXACT SOLUTION IS AVAILABLE. δ_{vd} IS APPROXIMATELY INVERSELY PROPORTIONAL TO SAFETY FACTOR USING CASE Ⓐ:

$\delta_{vd} = \frac{2.85}{1.90}(3.1) = \underline{4.6 \text{ IN.}}$

Ⓑ CONTINUOUS FOOTING WITH INCLINED LOAD



$C = 2.0$ KSF $\quad \phi = 0 \quad \gamma_T = 130$ PCF

STABILITY NUMBER, $N_o = \frac{\gamma_T H}{C} = 0.52$

FOR $D/B = 1$, $N_{cq} = 6.75$ $\}$ FROM

FOR $D/B = 0$, $N_{cq} = 4.3$ $\}$ FIG. 11-4

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

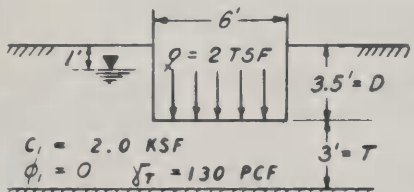
FOR $D/B = 3.5/6 = 0.58$, $q_{ult} = 9.1 + 0.58(4.9) = 11.9$ KSF

SAFETY FACTOR, $F_S = \frac{11.9}{4} = \underline{2.98}$

IMMEDIATE SETTLEMENT, USING CASE Ⓐ:

$\delta_{vd} = \frac{2.85}{2.98}(3.1) = \underline{3.0 \text{ IN.}}$

Ⓒ CONTINUOUS FOOTING PLACED AT TOP OF SLOPE



$C_1 = 2.0$ KSF

$\phi_1 = 0 \quad \gamma_T = 130$ PCF

$C_2 = 4.0$ KSF

$\phi_2 = 0 \quad \gamma_T = 130$ PCF

$C_2/C_1 = 4/2 = 2 \quad T/B = 3/6 = 0.5$

N_C (FROM FIG. 11-5) = 5.8

FOR $D/B = 3.5/6 \cong 0.5$, N_{CD}/N_C (FROM FIG. 11-5) = 1.15

$q_{ult} = C_1 N_{CD} + \gamma D = 2(5.8)(1.15) + [.13(1) + (.13 - .0625)(2.5)] = 13.6$ KSF

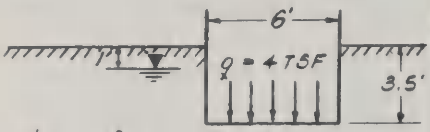
SAFETY FACTOR, $F_S = \frac{13.6}{4} = \underline{3.40}$

IMMEDIATE SETTLEMENT, USING CASE Ⓐ:

$\delta_{vd} = \frac{2.85}{3.40}(3.1) = \underline{2.6 \text{ IN.}}$

Ⓓ CONTINUOUS FOOTING ON TWO-LAYER FOUNDATION STRATA

FIGURE 11-6
Examples of Computation of Safety Factor and Immediate Settlement, Shallow Footings on Cohesive Soils



$\phi = 35°$ $\quad C = 0$

$\gamma_T = 130$ PCF

$q \begin{cases} DL = 2.5 \text{ TSF} \\ LL = 1.5 \text{ TSF} \end{cases}$

DEAD LOAD INCLUDES EFFECTIVE WEIGHT OF FOOTING WITH ASSUMED GROUND WATER LEVEL.

N_q (FROM FIG. 11-1) = 34

N_γ (FROM FIG. 11-1) = 40 (ROUGH)

K_{v1} (FROM FIG. 11-8) = 130 T/FT^3

FROM FIG. 11-2 AND $D \leq B$:

$$q_{ult} = CN_c + [\gamma_{SUB} D + (\gamma_T - \gamma_{SUB}) d] N_q + 0.5 \gamma_{SUB} B N_\gamma$$
$$= 0 + [.068 \times 3.5 + (.13 - .068) 1.0] 34 + 0.5 \times .0.68 \times 6 \times 40$$
$$= 18.52 \text{ KSF}$$

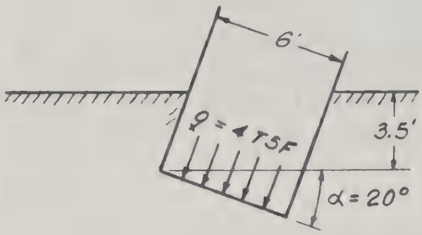
SAFETY FACTOR, $F_s = \dfrac{q_{ult}}{q} = \dfrac{18.52}{8} = \underline{2.32}$

IMMEDIATE SETTLEMENT

$$\delta_v = 2 \times \frac{4 q B^2}{K_{v1} (B+1)^2}$$

$$\delta_v = 2 \frac{4 \times 4 \times 6^2}{130 (6+1)^2} = .18 \text{ FT} = \underline{2.16} \text{ IN.}$$

Ⓐ CONTINUOUS FOOTING WITH VERTICAL LOAD



$\phi = 35°$ $\quad C = 0$

$\gamma_T = 130$ PCF

$q \begin{cases} DL = 2.5 \text{ TSF} \\ LL = 1.5 \text{ TSF} \end{cases}$

$N_{\gamma q}$ (FROM FIG. 11-3) = 60

$D/B = 3.5/6 = .58$

ASSUME GROUND WATER TABLE IS LOCATED AT GREAT DEPTH BELOW BASE OF FOOTING.

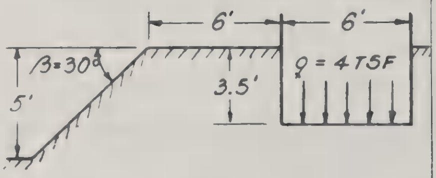
$$q_{ult} = CN_{cq} + \gamma \frac{B}{2} N_{\gamma q} = 0 + .130 \times \frac{6}{2} \times 60 = 23.4 \text{ KSF}$$

SAFETY FACTOR, $F_s = \dfrac{q_{ult}}{q} = \dfrac{23.4}{8} = \underline{2.93}$

IMMEDIATE SETTLEMENT, NO EXACT SOLUTION AVAILABLE. δ_v IS APPROXIMATELY INVERSELY PROPORTIONAL TO SAFETY FACTOR. USING CASE Ⓐ:

$$\delta_v = \frac{2.32}{2.93} (2.16) = \underline{1.71} \text{ IN.}$$

Ⓑ INCLINED CONTINUOUS FOOTING WITH INCLINED LOAD



$\phi = 35°$ $\quad C = 0$

$\gamma_T = 130$ PCF

$q \begin{vmatrix} DL = 2.5 \text{ TSF} \\ LL = 1.5 \text{ TSF} \end{vmatrix}$

$\dfrac{b}{B} = \dfrac{6}{6} = 1.0 \qquad \dfrac{D}{B} = \dfrac{3.5}{6} = .58$

$N_{\gamma q}$ (FROM FIG. 11-4)

$$N_{\gamma q} = \frac{10 + 40 + 38 + 120}{8} = 26$$

ASSUME GROUND WATER TABLE IS LOCATED AT GREAT DEPTH BELOW BASE OF FOOTING.

$$q_{ult} = CN_{cq} + \gamma \frac{B}{2} N_{\gamma q} = 0 + .13 \times \frac{6}{2} \times 26 = 10.2 \text{ KSF}$$

SAFETY FACTOR, $F_s = \dfrac{q_{ult}}{q} = \dfrac{10.2}{8} = \underline{1.28}$

IMMEDIATE SETTLEMENT, USING CASE Ⓐ

$$\delta_v = \frac{2.32}{1.28} (2.16) = \underline{3.91} \text{ IN.}$$

Ⓒ CONTINUOUS FOOTING PLACED BACK OF SLOPE

FIGURE 11-7

Examples of Computation of Safety Factor and Immediate Settlement, Shallow Footings on Cohesionless Soils

3. **SETTLEMENT.** Settlements produced by isolated individial footings are derived from two sources; deep-seated consolidation from volume change under combined loads of all foundation units, and immediate settlement from shear strain beneath individual footings.

a. Consolidation Settlements. See Chapter 6 for methods of consolidation analysis. Analysis may indicate need for replacing shallow spread foundations by deep foundations. In certain situations, time delayed consolidation of substrata will contribute the major settlements as follows:

(1) A structure supported by mat or individual footings founded on a compact bearing stratum underlain by soft, compressible material.

(2) A structure of low to medium area load placed directly on compressible fine grained material.

(3) A heavy structure placed within excavation in a thick stratum of compressible material.

b. Shear Strain Settlements. This deformation occurs immediately on application of load. Settlements under dead load during construction usually will not contribute to structural distress, but those caused by live load will affect the structure without the relief of plastic flow in structural members. In the following situations immediate settlement from shear strain under isolated footings comprises all, or a major portion, of total settlements:

(1) Individual footings on a thick stratum of coarse grained material whose rigidity increases with depth.

(2) Individual footings on a thick stratum of stiff or hard, preconsolidated clays where consolidation settlements are the result of recompression only.

c. Computations of Immediate Settlements. Where immediate settlements dominate, compute values by the procedure of Figure 11-8 for coarse grained soils, and Figure 11-9 for stiff or hard, fine grained soils. For examples of computation see Figures 11-6 and 11-7.

(1) Cohesionless Soils. For approximate analysis, use modulus of subgrade reaction K_{v1} in cohesionless soils given in the graph of Figure 11-8. For detailed study, determine K_{v1} from plate-bearing test. See Figure 4-4. Settlement formulas of Figure 11-8 are based on experience with individual footings of various sizes. If ground water level is as high as the base of footings, reduce modulus of subgrade reaction by one-half.

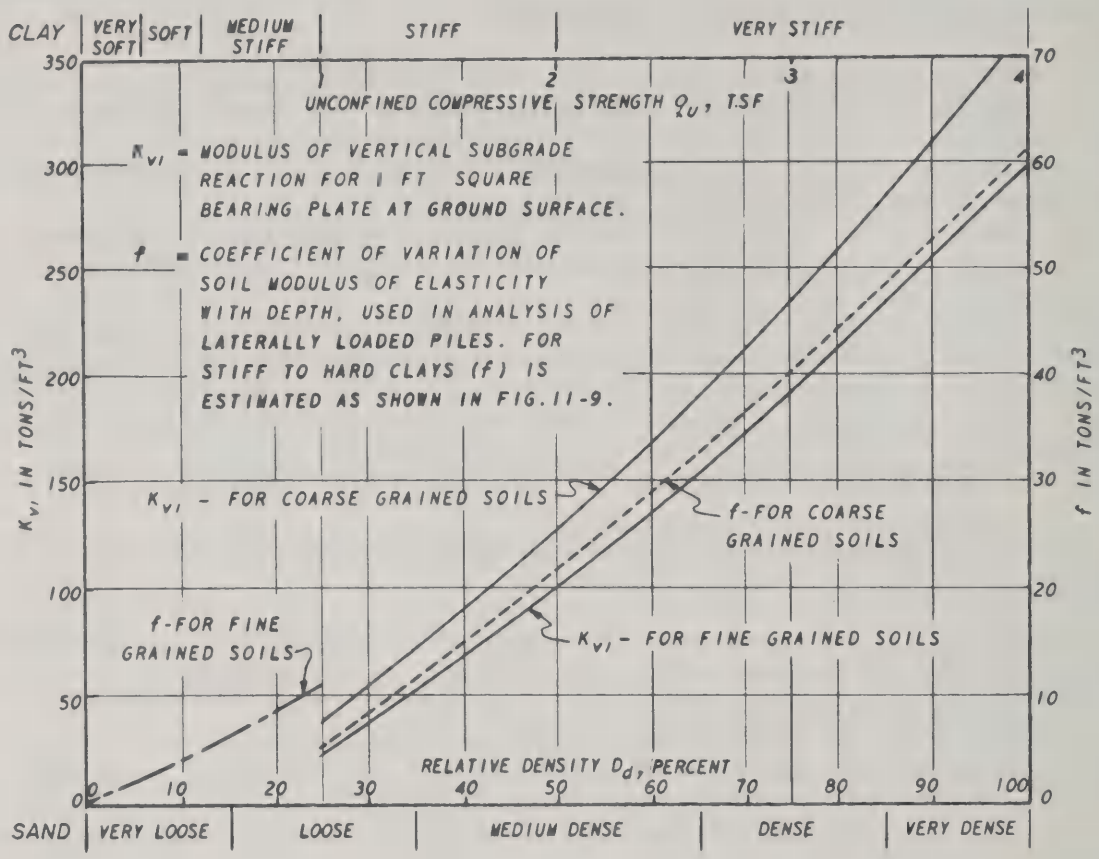
(2) Fine Grained Soils. For stiff to hard fine grained soils, compute settlements assuming elastic conditions with an allowance for embedment and shape of footing. In the general case of a soil with both cohesion and friction resistance, estimate settlement from shear strain from plate-bearing tests on plates of two different sizes. See Figure 4-4 for method of analysis.

d. Load Assumptions. Transient forces that are repeated periodically and may be of some duration include those of wind, snow, moving machinery, and equipment. Transient forces of extremely short duration occurring at irregular intervals include earthquakes, impact, blasting, and vibratory loads. Disregard transient forces in analyzing long-term settlements caused by volumetric consolidation. For computation of immediate settlements from shear strain under individual footings, include periodically repeated transient forces in live load.

4. **NOMINAL BEARING PRESSURES.** For preliminary estimates or when elaborate investigation of soil properties is not justified, use nominal bearings pressure of Table 11-1.

a. Utilization. These load intensities are intended to provide a reasonable safety factor against ultimate failure and to avoid detrimental settlements of individual footings. Nominal bearing pressures must be used with caution and verified, if possible, by performance of nearby structures.

b. Modifications of Nominal Bearing Pressures. See Table 11-2 for variations in allowable bearing pressure depending on footing size and position. Nominal bearing pressures may be unreliable for foundations on very soft to medium-stiff fine grained soils and should be checked by an estimate of theroetical bearing capacity. Where bearing strata are underlain by weaker and more compressible material or where



DEFINITIONS

σ_v = FOOTING SETTLEMENT
q = FOOTING UNIT LOAD IN tsf
B = FOOTING WIDTH
L = FOOTING LENGTH
D = DEPTH OF FOOTING BELOW GROUND SURFACE
E_s = MODULUS OF ELASTICITY OF SOIL
C_s = SHAPE COEFFICIENT OF LOADED AREA

FOR COARSE GRAINED SOILS AND VERY SOFT TO MEDIUM STIFF FINE GRAINED SOILS, E_s IS ASSUMED TO INCREASE LINERALLY WITH DEPTH ($E_s = fz$)

FOR STIFF TO HARD CLAYS E_s IS ASSUMED TO BE CONSTANT WITH DEPTH.

COARSE GRAIN SOILS

(MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH)

SHALLOW FOOTINGS $D \leqq B$

FOR $B \leqq$ 20FT:

$$\sigma_v = \frac{4qB^2}{K_{v1}(B+1)^2}$$

FOR $B \geqq$ 40FT:

$$\sigma_v = \frac{2qB^2}{K_{v1}(B+1)^2}$$

INTERPOLATE FOR INTERMEDIATE VALUES OF B

DEEP FOUNDATION $D \geqq 5B$

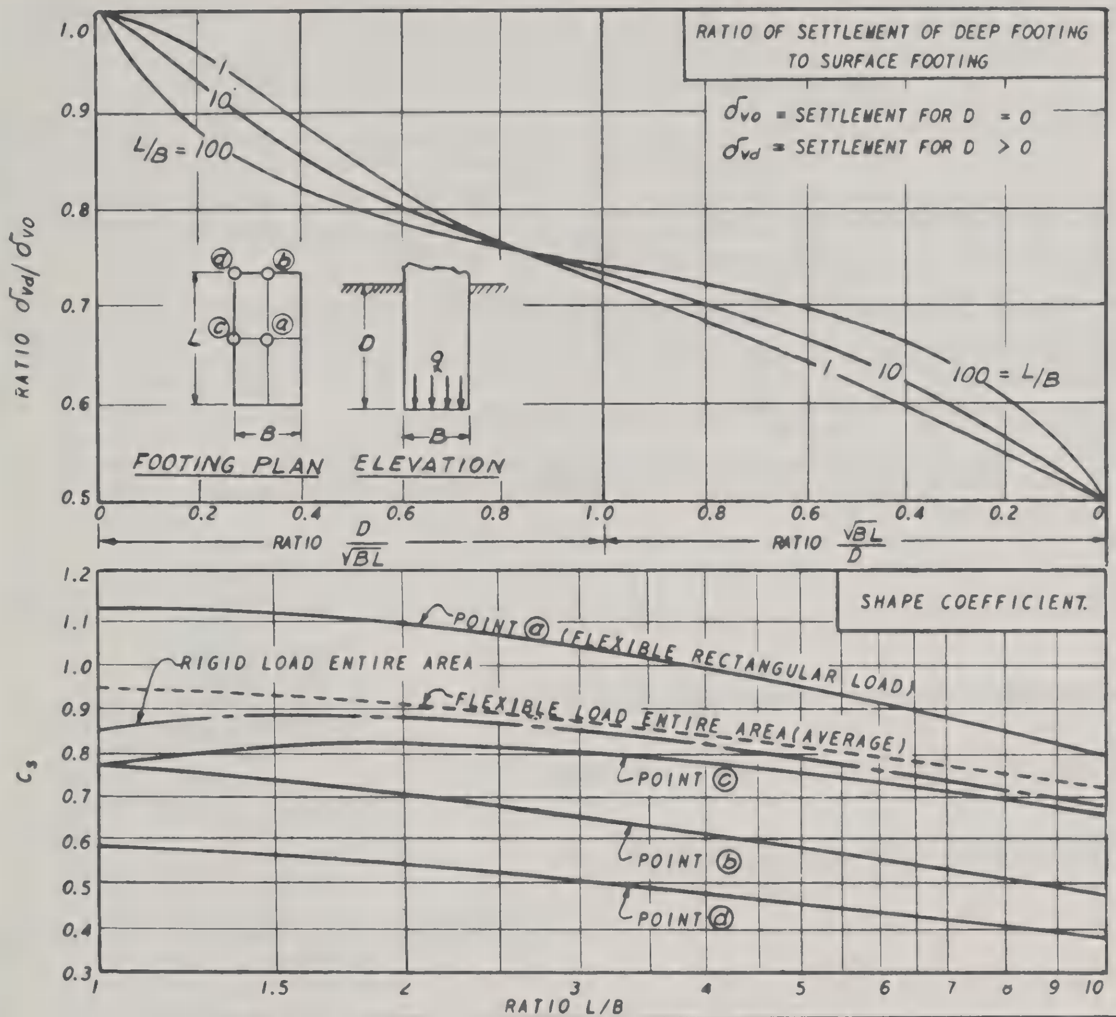
FOR $B \leqq$ 20FT:

$$\sigma_v = \frac{2qB^2}{K_{v1}(B+1)^2}$$

NOTES: 1: NONPLASTIC SILT IS ANALYZED AS COARSE GRAINED SOIL WITH MODULUS OF ELASTICITY INCREASING LINEARLY WITH DEPTH.

2. VALUES OF K_{v1} SHOWN FOR COARSE GRAINED SOILS APPLY TO DRY OR MOIST MATERIAL WITH THE GROUND WATER LEVEL AT A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING. IF GROUND WATER IS AT BASE OF FOOTING, USE $K_{v1}/2$ IN COMPUTING SETTLEMENT.

FIGURE 11-8
Immediate Settlement of Isolated Footings on Coarse Grained Soils



STIFF TO HARD FINE GRAINED SOILS

(MODULUS E_s IS CONSTANT WITH DEPTH)

SETTLEMENT FOR D = 0 (δ_{vo}):

$$\delta_{vo} = \frac{.75\, q\, B\, C_s \sqrt{L/B}}{E_s}$$

AND SINCE E_s = .72 K_{v1} (TONS AND FT. UNITS)

$$\delta_{vo} = \frac{1.05\, q\, B\, C_s \sqrt{L/B}}{K_{v1}}$$

SETTLEMENT FOR D > 0 (δ_{vd}):
COMPUTE δ_{vo} AND OBTAIN RATIO OF δ_{vd}/δ_{vo} FROM UPPER DIAGRAM.

POSITION OF GROUND WATER LEVEL DOES NOT AFFECT IMMEDIATE SETTLEMENT UNLESS SOIL IS PERMITTED TO SWELL. FOUNDATION MATERIAL IS ASSUMED UNIFORM TO A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING.

CONVERSION OF CONSTANT MODULUS E_s TO EQUIVALENT MODULUS E_s VARYING LINERALY WITH DEPTH FOR APPLICATION TO LATERALLY LOADED PILES IN STIFF TO HARD CLAYS.

PROCEDURE:

1. GIVEN A VALUE OF CONSTANT E_s, ASSUME A VALUE OF f.
2. COMPUTE DEPTH z TO POINT OF ZERO DEFLECTION OF PILE FROM FIGS. 13-4 OR 13-5.
3. RECOMPUTE f SO THAT VALUE OF fz = $2E_s$.
4. RECOMPUTE DEPTH z WITH REVISED f AND ALTER f AGAIN SO THAT fz = $2E_s$.

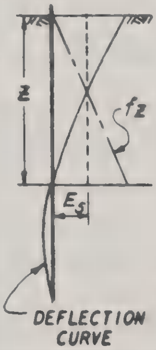


FIGURE 11-9
Immediate Settlement of Isolated Footings on Stiff to Hard Fine Grained Soils

TABLE 11-1
Nominal Values of Allowable Bearing Pressures for Spread Foundations

Type of bearing material	Consistency in place	Allowable bearing pressure tons per sq ft	
		Ordinary 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
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks).	Medium hard sound rock	30 to 40	35
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities.	Medium hard sound rock	15 to 25	20
Weathered or broken bed rock of any kind except highly argillaceous rock (shale).	Soft rock	8 to 12	10
Compaction shale or other highly argillaceous rock in sound condition.....	Soft rock	8 to 12	10
Well graded mixture of fine and coarse grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC).	Very compact	8 to 12	10
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP) ...	Very compact	7 to 10	8
	Medium to compact	5 to 7	6
	Loose	3 to 6	4
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	4 to 6	4
	Medium to compact	3 to 4	3
	Loose	2 to 3	2
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)..	Very compact	3 to 5	3
	Medium to compact	2 to 4	2.5
	Loose	1 to 2	1.5
Fine sand, silty or clayey medium to fine sand (SP, SM, SC).............	Very compact	3 to 4	3
	Medium to compact	2 to 3	2
	Loose	1 to 2	1.5
Homogeneous inorganic clay, sandy or silty clay (CL, CH)..............	Very stiff to hard	3 to 6	4
	Medium to stiff	1 to 3	2
	Soft	.5 to 1	.5
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH) ...	Very stiff to hard	2 to 4	3
	Medium to stiff	1 to 3	1.5
	Soft	.5 to 1	.5

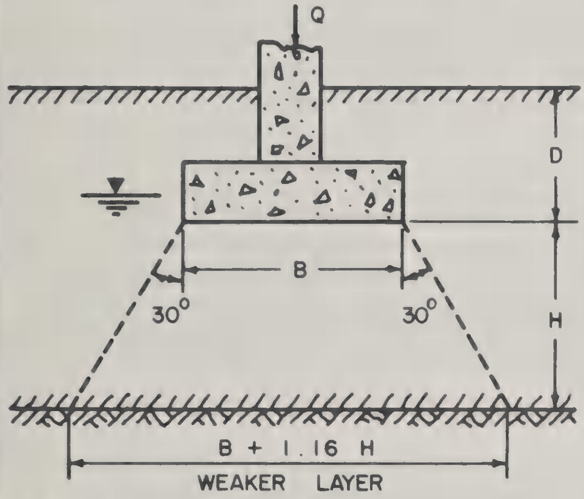
Notes:
1. Variations of allowable bearing pressure for size, depth and arrangement of footings are given in Table 11-2.
2. Compacted fill, placed with control of moisture, density, and lift thickness, has allowable bearing pressure of equivalent natural soil.
3. Allowable bearing pressure on compressible fine grained soils is generally limited by considerations of overall settlement of structure, Table 6-1.
4. Allowable bearing pressure on organic soils or uncompacted fills is determined by investigation of individual case.

compressibility of subsoils is constant with depth, analyze consolidation settlement of the entire foundation.

5. PROPORTIONING INDIVIDUAL FOOTINGS. Where significant compression will not occur in strata below a depth equal to the distance between footings, proportion size of individual footings to give equal settlement; use formulas for immediate settlement in Figures 11-8 and 11-9. Where significant consolidation settlements may occur below this depth, select footing size on the basis of the safety factor against ultimate failure as a first trial. Analyze overall consolidation settlements for the combined effect of these individual footings. In this case, settlements are controlled by the combined stresses of all foundation units and may be little affected by alteration of individual footing areas.

TABLE 11-2
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 nominal values of allowable bearing pressure given in Table 11-1. 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 11-1.
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 in. 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 Item 4.
6. Apply the nominal bearing pressures of the three categories of hard or medium hard rock shown on Table 11-1 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 ft 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:



Q = applied load, not including weight of foundation itself.

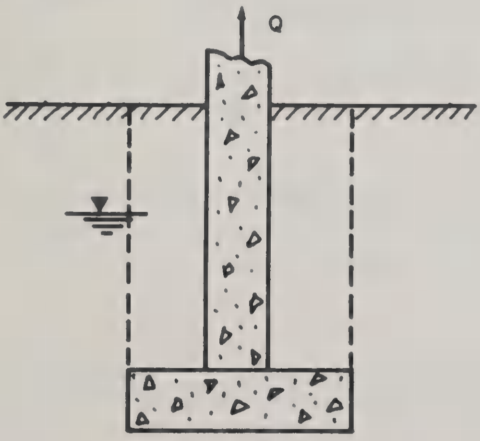
L = length of foundation.

$(B + 1.16H)\ (L + 1.16H)$ = area stressed in weaker layer.

$$\frac{Q}{(B + 1.16H)\ (L + 1.16H)} \leqq \text{nominal value of allowable bearing pressure.}$$

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 force.

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.

$$\text{Safety Factor} = \frac{W}{Q} \geqq 2$$

6. UPLIFT CAPACITY. Analysis of shallow foundations to resist uplift are given in Figures 11-10 and 11-11.

a. Rock Anchorages. Resistance to direct uplift of tower legs, guys, and antennas may be provided by reinforcing bars grouted in rock. In the absence of pullout tests, determine uplift resistance by empirical formulas of Figures 11-10, 11-11, 11-12, and Table 11-2. 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 strength may be increased by using washers, rock bolts, deformed bars, or splayed bar ends.

b. Anchorage in Soil. For sustained uplift on footing, see Table 11-2. Uplift is considered to be resisted by weight of footing plus soil directly overlying footing only. Transient uplift from live loads applied to footings, piers, posts, or anchors is analyzed as shown in Figure 11-11. Tower guy anchorage in soil is analyzed in Figure 11-12.

Section 3. MAT FOUNDATIONS

1. APPLICATIONS. Mat foundations are 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 difficult to define; if shallow shear strain settlements predominate and the mat would equalize differential settlements; or if resistance to hydrostatic uplift is required.

2. STABILITY REQUIREMENTS. Where mats are placed on coarse grained soils whose rigidity increases with depth, the safety factor against overall shear failure exceeds that of footings and its adequacy need not be investigated in detail. For fine grained, cohesive soils, analyze overall stability by Figure 11-5. For mats on loose, cohesionless, silty fine sands or loessial silts and fine sands, compact the foundation in depth, use methods of Chapter 15. In cohesive soils of constant strength, lower the base elevation of the mat to increase overall safety factor.

3. DESIGN OF MAT FOUNDATIONS. Design is based on the theory for beams or plates on elastic foundations. An outline of methods is presented herein to show their applicability in various problems. For information on detailed procedures, see Hetenyi, *Beams on Elastic Foundation* (Bibliography). Elastic analyses are suitable particularly for foundations on coarse grained soils where rigidity increases with depth and settlements are derived principally from strain within shallow depths.

a. Two-dimensional Problems. For wall or track footings or mat foundations subjected to plane strain, such as drydocks with long walls and linear blocking loads, perform elastic analysis by method of beams on elastic foundations. Specific procedures are located as follows:

(1) For definitions and procedures, see Table 11-3.

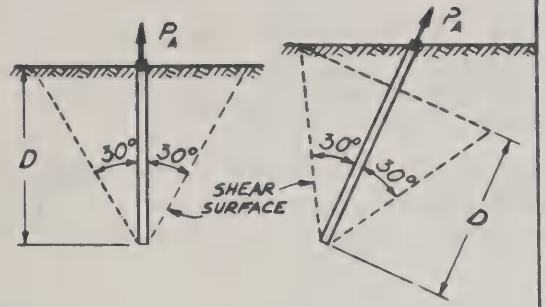
(2) For formulas for shear, moment, and deflection under various applied loads, see Figure 11-13.

(3) For functions used for computation of shear, moment, and deflection, see Figure 11-14.

(4) 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 equidimensional mat, analyze stresses by method of plates on elastic foundations. Detailed procedures are located as follows:

(1) See Table 11-4 for definitions and procedures.



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 $P_A = Af_s$

$$b_{rqd} = \frac{P_A}{\text{BAR PERIMETER} \times D}$$

TESTS INDICATE THAT FOR BAR IN ORDINARY FRACTURED ROCK NEAR THE SURFACE:

MINIMUM D(FT) = $(1.25)\sqrt{P_A}$ (KIPS)

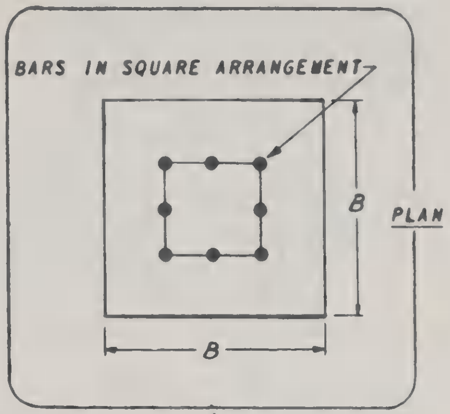
AT THIS DEPTH C_{all} = 0.3 KSF AND SHOULD NOT BE TAKEN GREATER THAN THIS VALUE WITHOUT PULLOUT TESTS.

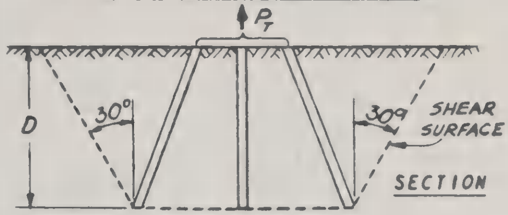
SPACING OF BARS IN PLAN SHOULD EXCEED 1.2D.

EXAMPLE:

GIVEN: $P_A = 20^K$ FOR 1 IN. SQ BAR
MINIMUM D = $1.25\sqrt{20}$ = 5.6 FT.
BAR SPACING = 1.2(5.6) = 6.7 FT.

$$b_{rqd} = \frac{20{,}000}{4(5.6)(12)} = 74 \text{ PSI}$$





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
$P_T = NAf_s$

$$b_{rqd} = \frac{P_T}{\text{BAR PERIMETER} \times ND}$$

TESTS INDICATE THAT FOR BAR GROUP IN ORDINARY FRACTURED ROCK NEAR THE SURFACE:

MINIMUM D(FT)

$$D = \frac{-4.6BC_{all} + \sqrt{21.2B^2(C_{all})^2 + 10.7C_{all} \times NAf_s}}{5.34\,C_{all}}$$

AT THIS DEPTH C_{all} = 0.3 KSF AND SHOULD NOT BE TAKEN GREATER THAN THIS VALUE WITHOUT PULLOUT TESTS.

EXAMPLE:

GIVEN $P_T = 80^K$ USE 4 - 1 IN SQ. BARS.
B = 4.5 FT, f_s = 20 KSI
MIN. D: WITHOUT TESTS:

$$D = \frac{-4.6 \times 4.5 \times 0.3 + \sqrt{21.2 \times 4.5^2 \times 0.3^2 + 10.7 \times 0.3 \times 4 \times 1 \times 20.0}}{5.34 \times 0.3}$$

= 6.9 FT

$$b_{rqd} = \frac{80{,}000}{(4)(4)(6.9)(12)} = 60 \text{ PSI}$$

FIGURE 11-10
Capacity of Anchor Rods in Fractured Rock

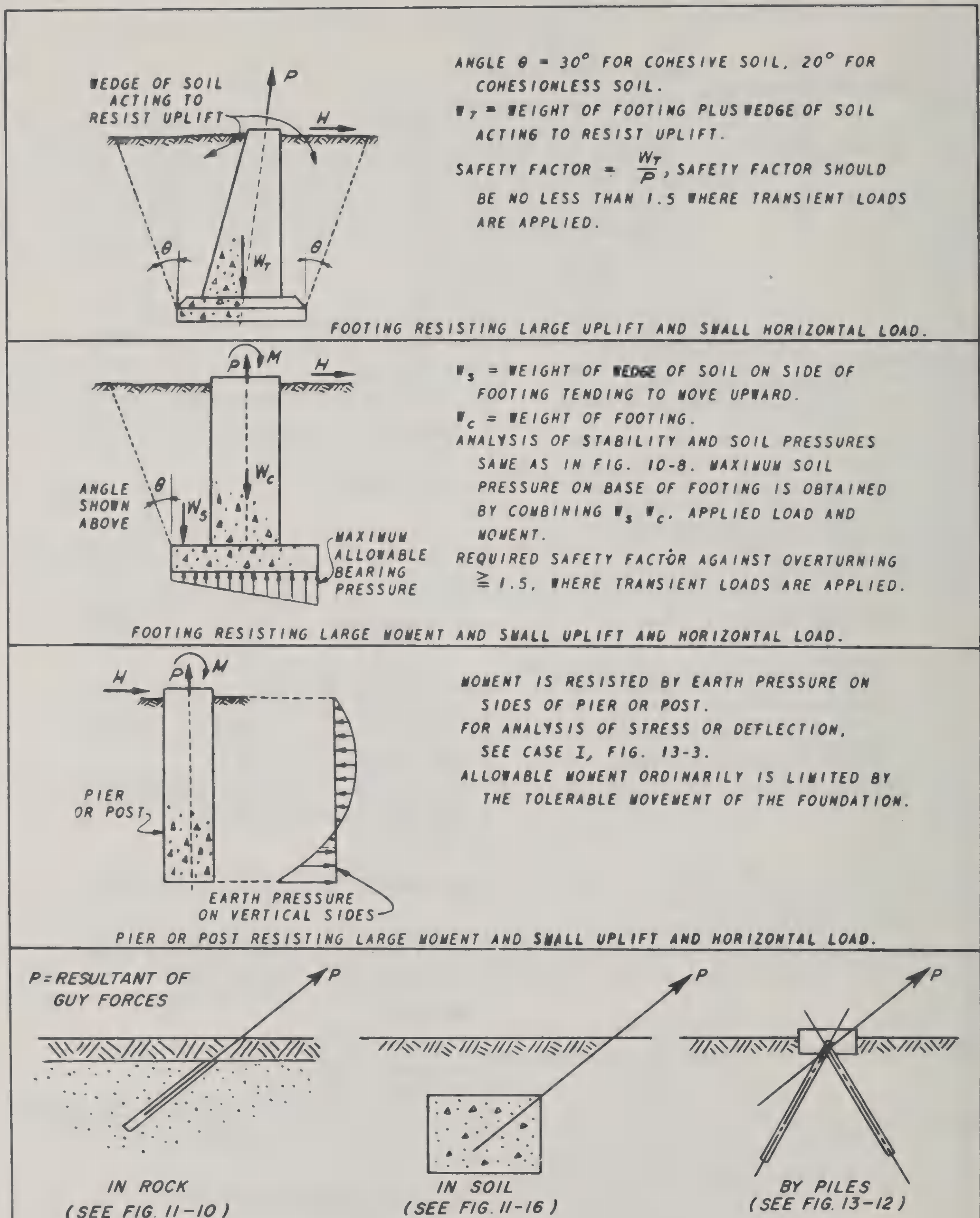
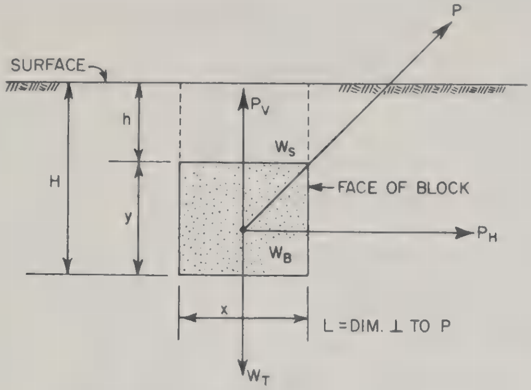


FIGURE 11-11
Resistance of Footings and Anchorages to Combined Transient Loads



P = RESULTANT OF MAXIMUM GUY FORCES
P_V, P_H = COMPONENTS OF P
W_T = WEIGHT OF BLOCK + SOIL ON BLOCK
$\qquad W_B \qquad W_S$
x, y, L = BLOCK DIMENSIONS
γ = UNIT WEIGHT OF SOIL, p.c.f.
$W_S = x \cdot L \cdot h \cdot \gamma$
P_p = TOTAL PASSIVE PRESSURE LBS/L.F.
ϕ = ANGLE OF INTERNAL FRICTION
C = COHESION, p.s.f.

1. RESISTANCE TO VERTICAL FORCE

SAFETY FACTORS IN VERTICAL DIRECTION :
[USE TOTAL UNIT WEIGHTS ABOVE WATER TABLE,
" BOUYANT " " BELOW " "]

$$\left\{ \begin{array}{l} \dfrac{W_T}{P_V} \geqq 1.5 \\ \dfrac{W_B}{P_V} \geqq 1.0 \end{array} \right.$$

2. RESISTANCE TO HORIZONTAL FORCE

SAFETY FACTOR IN HORIZONTAL DIRECTION :

a. SEE SECTION 4, CHAPTER 5 FOR P_p COMPUTATIONS
b. PASSIVE RESISTANCE CONSIDERED ON FACE OF BLOCK (AREA y x L) ONLY.

$$\left\{ \dfrac{P_P}{P_H} = 1.5 \right.$$

NOTES: BACKFILL SHALL BE COMPACTED AS SPECIFIED IN TABLE 9-3

EXAMPLE:

$\phi = 30°$; C = 0
WATER TABLE AT 5' DEPTH
$P = 40^K$; $P_V = 27^K$; $P_H = 30^K$
γ = 110 pcf, DRY
TRY BLOCK x, y, L = 6', 5', 8'
h = 2'
KEEP P_H AT 1/2 TO 2/3 BLOCK DEPTH BY VARYING x & y

VERT: W_B (ABOVE W.T.) = 6' x 8' x 3' x 150 pcf = 21,600#
W_B (BELOW W.T.) = 6' x 8' x 2' x 87.5 pcf = 8,400
W_B = 30,000
W_S = 6' x 8' x 2' x 110 pcf $\quad W_S$ = 10,500
W_T = 40,500#

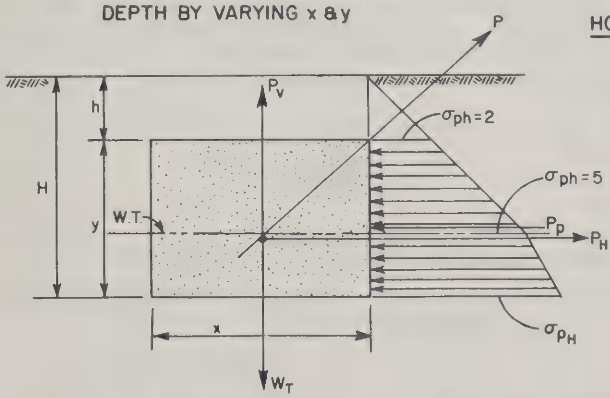
S.F. CHECK $\left\} \dfrac{W_T}{P_V} = \dfrac{40.5^K}{27^K} = 1.5 \quad \dfrac{W_B}{P_V} = \dfrac{30^K}{27} = 1.1 \right.$

$\therefore$ OK VERT.

HORIZ: FROM FIG. 5-12, WITH $\phi = 30°$, $\beta = 0°$: $K_p = 3.0$
FROM FIG. 5-10 :

$$\sigma_{p_{h=2}} = K_p \gamma h = 3.0 \times 110 \times 2 = 660$$
$$\sigma_{p_{h=5}} = K_p \gamma h = 3.0 \times 110 \times 5 = 1650$$
$$\sigma_{p_H} = K_p \gamma_{SUB} H + \gamma h_{W.T.} = 3.0 \times 60 \times 7 + 550 = 1800$$
$$P_p = 1/2 \cdot 3 (660 + 1650) L + 1/2 \cdot 2 (1650 + 1800) L$$
$$= 3465 L + 3450 L = 6915 \times 8 = 55,200\#$$
$$\frac{P_p}{P_H} = \frac{57^K}{30^K} = 1.83 > 1.5 \text{ S.F.}$$

$\therefore$ OK HORIZ.



MAKE ADDITIONAL TRIALS VARYING h, x, y, L

FIGURE 11-12
Tower Guy Anchorage in Soil by Concrete Deadman

Metz Reference Roo
University of Illinoi
B106 NCEL
208 N. Romine Street
na, Illinois 6180

TABLE 11-3
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	=	Characteristic of the system of beam and supporting soil $= \sqrt[4]{\dfrac{K_b b}{4EI}}$

Procedure for Analysis:

1. Determine E and establish K_{v1} from Fig. 11-8 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 contact pressure as for a rigid footing. Compute shear and moment in beam by simple statics.

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 Fig. 11-12. Determine in this way the shears 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, solutionsfor 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 determined from the infinite beam formulas. See Hetenyi, Beams on Elastic Foundation (Bibliography) for formulas for moments and shears in end loaded beam of finite length.

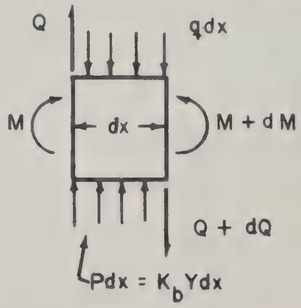
Group 3, long beams: $\lambda\ell > \pi$. End condition at distant end has negligible influence on moment and shear in the interior of the beam. Consider beam as extending an infinite distance away from loaded end. Compute moment and shear caused by interior loads by formulas for infinite beam, top panel of Fig. 11-12. Compute moment and shear for loads applied near the beam ends by formulas for semi-infinite beam, bottom panel of Fig. 11-12. 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 Fig. 11-12 from Fig. 11-13.

Sign Convention:

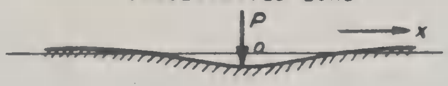
Consider infinitely small element of beam between two vertical cross sections at a distance d_x apart.

+Q	=	Upward acting shear force to left of section.
+M	=	Clockwise moment acting from the left of the section.
+y	=	Downward deflection.



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

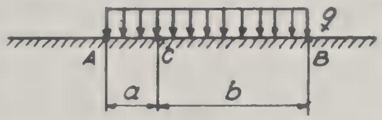
DEFLECTION: $y = \frac{M_0 \lambda^2}{K} B_{\lambda x}$

MOMENT: $M = \frac{M_0}{2} D_{\lambda x}$

SHEAR: $Q = -\frac{M_0 \lambda}{2} A_{\lambda x}$

UNIFORMLY DISTRIBUTED LOAD

POINT C IS UNDER LOAD

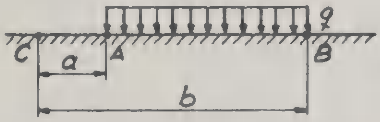


DEFLECTION: $y_C = \frac{q}{2K}(2 - D_{\lambda a} - D_{\lambda b})$

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

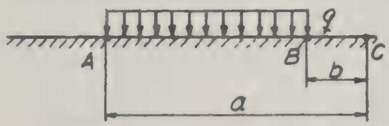


DEFLECTION: $y_C = \frac{q}{2K}(D_{\lambda a} - D_{\lambda b})$

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



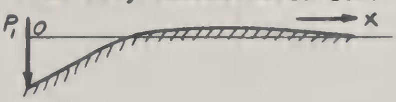
DEFLECTION: $y_C = -\frac{q}{2K}(D_{\lambda a} - D_{\lambda b})$

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

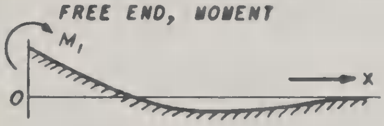


DEFLECTION: $y = \frac{2P_1\lambda}{K} D_{\lambda x}$

MOMENT: $M = -\frac{P_1}{\lambda} B_{\lambda x}$

SHEAR: $Q = -P_1 C_{\lambda x}$

FREE END, MOMENT

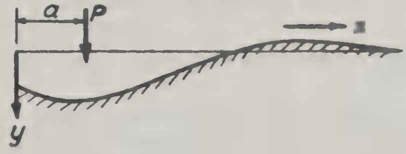


DEFLECTION: $y = -\frac{2M_1\lambda^2}{K} C_{\lambda x}$

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 11-13
Computation of Shear, Moment, and Deflection Beams on Elastic Foundation

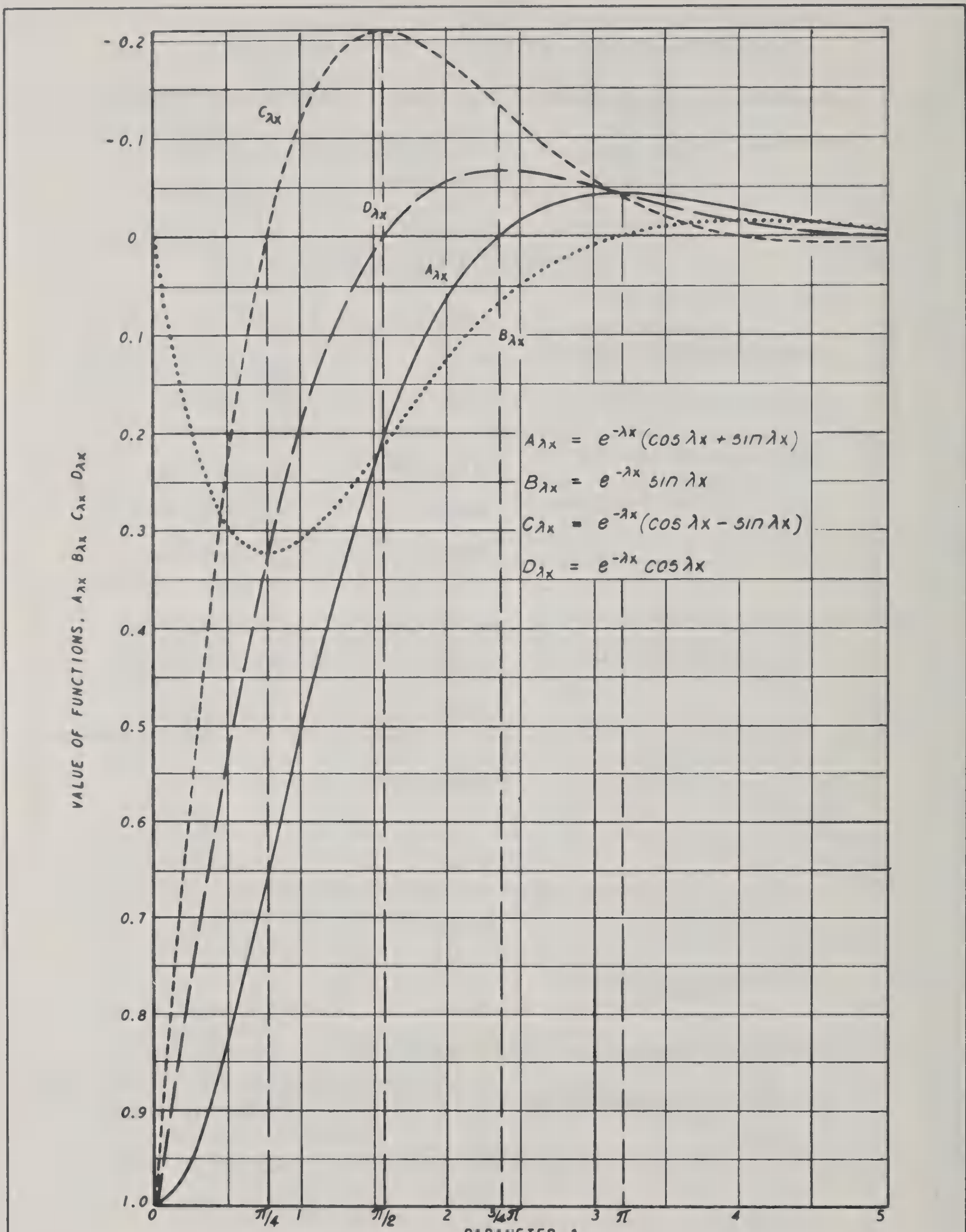


FIGURE 11-14
Functions for Shear, Moment, and Deflection Beams on Elastic Foundations

TABLE 11-4
Definitions and Procedures, Mats on Elastic Foundations

Definitions:

r	=	Distance of point under investigation from point load along radius.
M_r, M_t	=	Radial and tangential moments (polar coordinates) for a unit width of mat.
Q_r, Q_t	=	Radial and tangential shear.
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 mat K_b as follows:

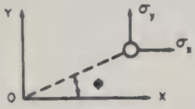
   For cohesive soils: $K_b = K_{v1}/b$, For cohesionless soils: $K_b = K_{v1}\left(\frac{b+1}{2b}\right)^2$

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, $D = \dfrac{Eh^3}{12(1-\mu^2)}$.
5. Calculate radius of effective stiffness: $L = \sqrt[4]{\dfrac{D}{K_b}}$.
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:

$$M_r = -\frac{P}{4}\left[Z_4(\xi) - (1-\mu)\frac{Z_3'(\xi)}{\xi}\right] \qquad \xi = \frac{r}{L}$$

$$M_t = -\frac{P}{4}\left[\mu Z_4(\xi) + (1-\mu)\frac{Z_3'(\xi)}{\xi}\right] \qquad y = \frac{PL^2}{4D} Z_3(\xi)$$

To convert radial and tangential moments to rectangular coordinates, use the following relationships:



$$M_x = M_R \cos^2\varphi + M_T \sin^2\varphi$$

$$M_y = M_R \sin^2\varphi + M_T \cos^2\varphi$$

Determine functions $Z_3(\xi)$ $Z_3'(\xi)$ and $Z_4(\xi)$ from Fig. 11-14.

8. To determine moments or deflections from a combination of interior column loads, superpose the effects from individual column loads at points under consideration.
9. When edge of mat is located within the radius of influence of the individual column loads, apply the following correction:

   a. Calculate moments and shears that occur perpendicular to the edge of mat within the radius of influence of the column load by analyzing the location of the edge in infinite mat formulas.

   b. Apply redundant moments and shears of opposing signs at the edge of the mat. Determine moments and shears produced within the mat by the redundants by analyzing a series of beams on elastic foundations positioned perpendicular to the edge, applying formulas of the bottom panel of Fig. 11-12. Utilize a similar procedure for large openings in the interior of the mat. Superpose these moments to moments computed in Step 8.

10. When superstructure loads are distributed through deep foundation walls, use the following procedure:

    a. Estimate an approximate distribution of superstructure loads as a line load along the wall.

    b. Divide the mat into a series of strips 1 ft wide perpendicular to the foundation wall with the line load acting at the end. Analyze the strips as beams on elastic foundations using formulas of the top panel of Fig. 11-12 for interior foundation walls and formulas of the bottom panel of Fig. 11-12 for foundation walls at edge of mat.

    c. Superpose moments and shears determined from this analysis with those obtained from interior column loads on the mat.

(2) See Figure 11-15 for functions used for computation of shear, moment, and deflection.

(3) Superpose shear, moment, or deflection produced by separate loads to obtain the effect of combined loads.

c. Special Problems. Elastic analysis must be modified for thick and extremely rigid mats or mats with a rigid superstructure (e.g., a silo) where uniform settlement will result, and where subsoils become more compressible with depth, for settlement, which will be caused by deep compression, more or less independent of the mat load arrangement. In the first case, foundation contact pressures must be estimated by judgment. In cohesionless soils, contact pressures at the edge of mat may be somewhat less than beneath the center. In cohesive soils, assume, for a first trial, contact pressures at mat edge one-third greater than average, decreasing to minimum pressures at mat center. Compute settlements for these assumed contact pressure diagrams and alter pressures as necessary to obtain uniform settlement. In the second case, assume a probable contact pressure diagram and compute the profile of consolidation settlements. Apply to the mat the angle changes produced by this settlement profile and determine resulting moments in the mat. Add these moments to those computed from the elastic analysis to determine the overall effect.

Section 4. GENERAL REQUIREMENTS

1. LIMITATIONS FOR DEPTH AND POSITION. See Table 11-5 for requirements governing depth, size, and position of spread foundations. For detailed requirements concerning frost effects, see Chapter 16 and NAVFAC DM-9.

2. FOUNDATIONS ON SWELLING SOILS. See Table 6-2 for summary of various swelling problems and possible treatment.

a. Potential Swelling Conditions. If surface clays above the water table have a PI greater than about 22 (CH clays) and relatively low natural water content, potential swelling must be considered. Swell is accentuated in arid climates with a deficiency of rainfall over evaporation and where the ground water table is low. Mottled, fractured, or slickensided clays, showing evidence of past desiccation, are particularly troublesome. To determine magnitude of potential swell, perform laboaratory swell tests, determining expansion after saturation under loads to be applied in situ. See Figure 6-5.

b. Eliminating Swell. Where economically feasible, remove potentially swelling foundation soils and replace with compacted fill of cohesionless or nonswelling materials. See Figure 6-5 for analysis of undercut required. If material cannot be undercut, design footings with unit loads large enough to resist swelling pressures without exceeding bearing capacity. Support the floor aboveground. Consider also spread footings or drilled and underreamed caissons founded below the zone of active swelling. Support the floor aboveground. Design the shafts for 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. 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.

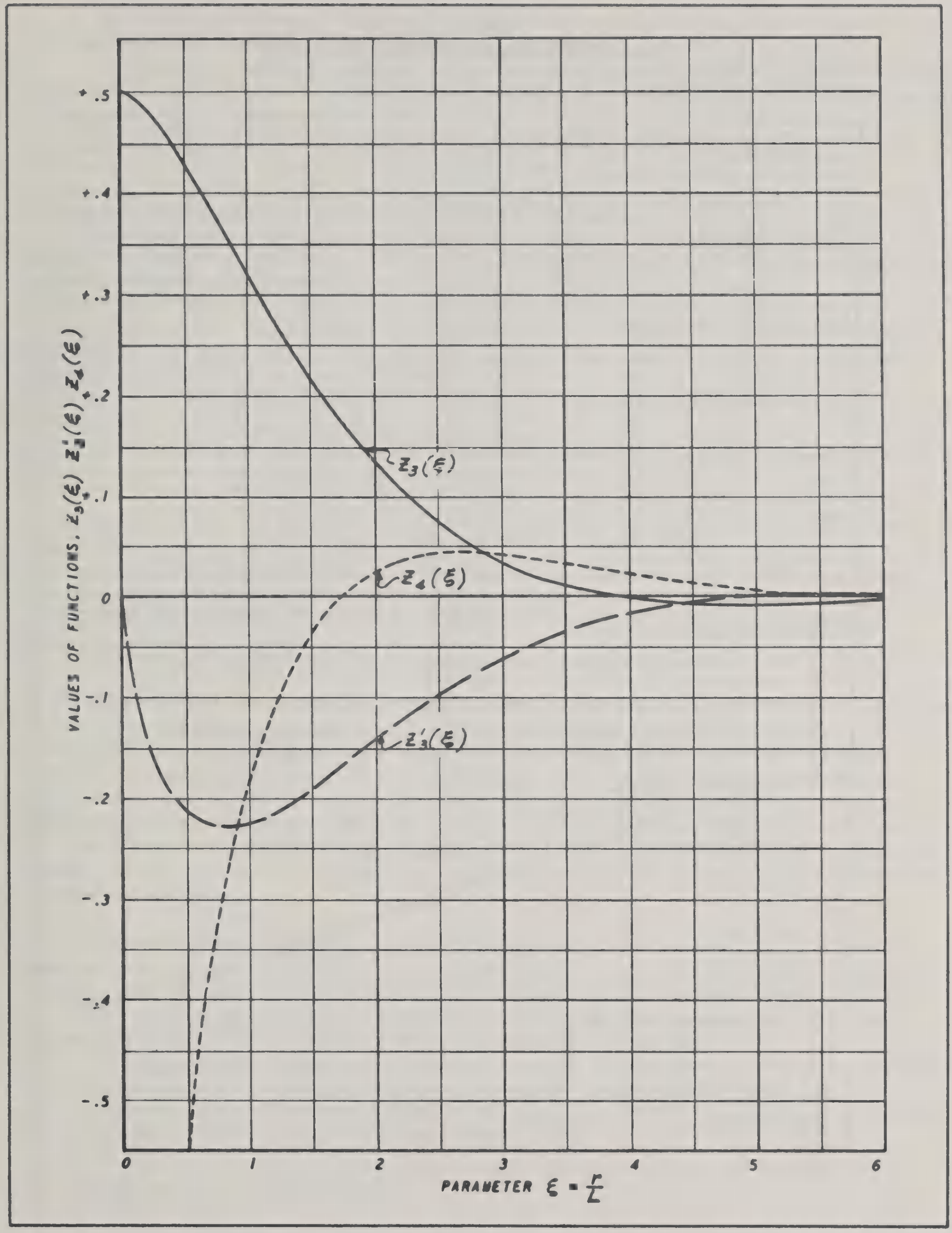


FIGURE 11-15
Functions for Shear, Moment, and Deflection Mats on Elastic Foundation

TABLE 11-5
General Requirements For Spread Foundations

1. The base of spread foundations shall be located below the depth to which the soil is subject to seasonal volume changes caused by alternate wetting and drying or within which frost may cause a perceptible heave.

2. Where foundations are to be placed in an excavation below the water table, provide for drawdown of water levels so that the work can be done in the dry and no piping, boiling, or heaving occurs in soils that will support the foundations. Ordinarily this requires lowering the ground water within the foundation area to an elevation no higher than 2 ft below subgrade.

3. For foundations supported in fine grained soil that would be disturbed and softened by construction activities upon it, provide a working mat at subgrade of lean concrete or cohesionless, coarse grained materials.

4. For clays or shales that will expand and soften with release of overburden, place working mat at subgrade immediately after completion of excavation. Provide surface drainage facilities to prevent collection of water in excavation as it nears the subgrade.

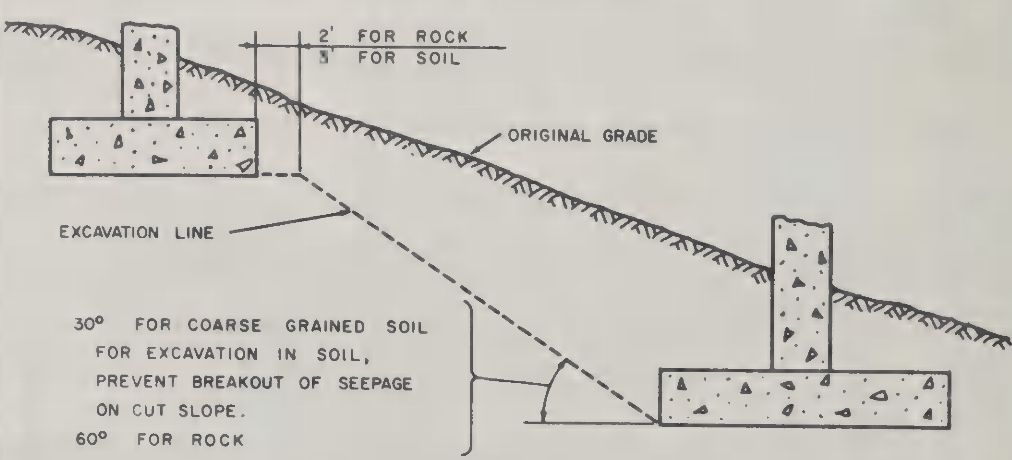
5. Where adjacent footings in the same structure bottom on materials of substantially different bearing quality, such as medium compact soil and rock, provide a cushion of yielding material beneath the footing on the harder foundation. For the footings on rock, place an 18-in.-thick layer of uncompacted sand above the bearing surface in rock. Where practical, consider separating the portions of the structure on dissimilar materials by expansion joints.

6. For ordinary warehouse floor slabs on grade, compact the subgrade in natural soils to a depth of 8 in., as specified in Table 9-3. To avoid dampness on the warehouse floor, provide a vapor barrier of plastic sheeting at base of slab. In addition, where ground water table is within about 8 ft of base of slab provide a base course 8 in. thick of clean, coarse grained material beneath the floor slab, if underlying soils are fine grained or are silty fine sands with considerable capillary potential.

7. For warehouse floors supporting wheel loads, provide floor slab and base course plus subgrade compaction conforming to pavement requirements of NAVFAC DM-5.

8. For requirements for structural fills to support spread foundations, see Table 9-4.

9. Adjacent footings at different bearing levels shall be separated as follows:



This requirement shall not apply where adequate lateral support by bracing is provided for the material beneath the higher footing. Where the higher footing is supported on cohesive soils, evaluate its bearing capacity in accordance with Fig. 11-4.

c. Minimizing Swell Effects. Where it is not economically feasible to remove swelling materials or to support foundations below swelling depths, minimize the effects as follows:

(1) 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.

(2) For concrete floor slabs placed directly on potentially expansive clays, provide expansion joints so the floor can move freely from the structural frame.

(3) For foundations on fill materials containing plastic fines, place fill at moisture content above optimum with density no higher than required for strength and rigidity.

(4) 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.

3. UNDERDRAINAGE AND WATERPROOFING. See Table 11-6 for general requirements for waterproofing, dampproofing, and waterstops. For basements below ground, two general schemes are employed as follows:

(1) Where the permanent water table is above the top of basement slab, provide pressure resistant slab or relieve uplift pressures by underdrainage.

(2) Where the water table is deep but infiltration of surface water dampens backfill surrounding basement, dampproof walls and slabs.

a. Relieved Slabs. For basements at considerable depth below ground water level, it is usually economical to provide pressure relief beneath the foundation slab. See case B and C, Figure 11-16. If pervious materials of great depth underlie the foundation level, include a wall drain and drainage course beneath the slab. See Chapter 8 for filter requirements and drain spacing. If foundation walls can be carried economically to underlying sound rock or impervious stratum, omit wall drains. Arrange sumps for drainage discharge to avoid aerating drainage course.

b. Pressure Slabs. In general, the choice between relieved or pressure 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 ground water, a pressure slab to resist maximum probable hydrostatic uplift usually is economical. See Case A, Figure 11-16. For basement occupancy of high type, provide a drainage course of free-draining material beneath the finished floor to intercept leakage into the basement. Drainage material should be sound, clean gravel or crushed stone graded between 3/4 and 2 in., compacted by two or three coverages of vibrating base plate compactor. Open joint drain pipe should be added beneath slabs of large plan dimension. Provide waterstops at the construction joints between pressure slab and wall.

c. 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 may interrupt capillary action, but will not prevent movement of water vapor through slabs.

(1) Membrane Waterproofing. Apply membrane waterproofing for basements utilized for routine purposes where appearances are unimportant and some dampness is tolerable.

(2) 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 the most effective guarantee against dampness and moisture.

4. FOUNDATIONS ON COMPACTED FILL.

a. Utilization. Compacted fills may be used beneath structures for the following purposes:

(1) To raise the general grade of the structure or to replace unsuitable foundation soils that must be removed.

TABLE 11-[illegible]

Requirements for Foundation Waterproofing and Dampproofing

Type	Materials	Workmanship	Applicability	Remarks
Waterproofing: Membrane....	Coal tar, straight run, pitch Type B; coal tar saturated felt weighing at least 15 lb per 100 sq ft; coal tar primer; protective coat of 1-in. fiberboard or 2-in. lean concrete.	Concrete surfaces to be dry and free from dust, dirt, grease, oil, and other coatings. Remove rough edges. Apply primer and alternately install pitch and layers of felt. Provide fiberboard protective coat for vertical faces. Provide 2-in. concrete protection over horizontal surfaces. Use minimum 3-ply application for dampproofing, 5-ply application against hydrostatic pressures.	Use on exterior wall surfaces, over roofs of 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.
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 coats shall be applied in 2 coats that together total between 5/8 to 3/4 in. in thickness. Floors to have one coat of 1 in. in thickness. All surfaces are to be floated with wood float and hand finished by steel troweling.	Used on exposed interior surfaces of wall, floors, and occasionally on ceilings where the ceiling is exposed to 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: Interior faces.	Coating consisting of finely divided iron mixed with sand, cement, and an 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 ground water levels, or under hydrostatic heads of only several feet.	Lower cost. If appearance of interior surfaces is important, use cement plaster waterproofing.
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 in. thickness.	Concrete surfaces to be dry and free from dust, dirt, grease, oil, or other coatings before application.		

Materials and Dimensions	Workmanship	Remarks
Waterstops:		
Copper: 20 oz annealed soft copper with No. 4 copper wire rolled in each edge, 12-in. min. width.	All joints to be soldered..........	Easily punctured or ripped.
Rubber: Minimum 9 in. wide, 3/8 in. thick	All joints to be vulcanized........	Hard to splice.
Plastic (Polyvinylchloride): Minimum 9 in. wide, 3/16 in. thick.	Materials to conform to ASTM D412, D624, D676. All joints to be heat fused.	Generally most convenient for handling and splicing. Performance for long-term use not completely known.

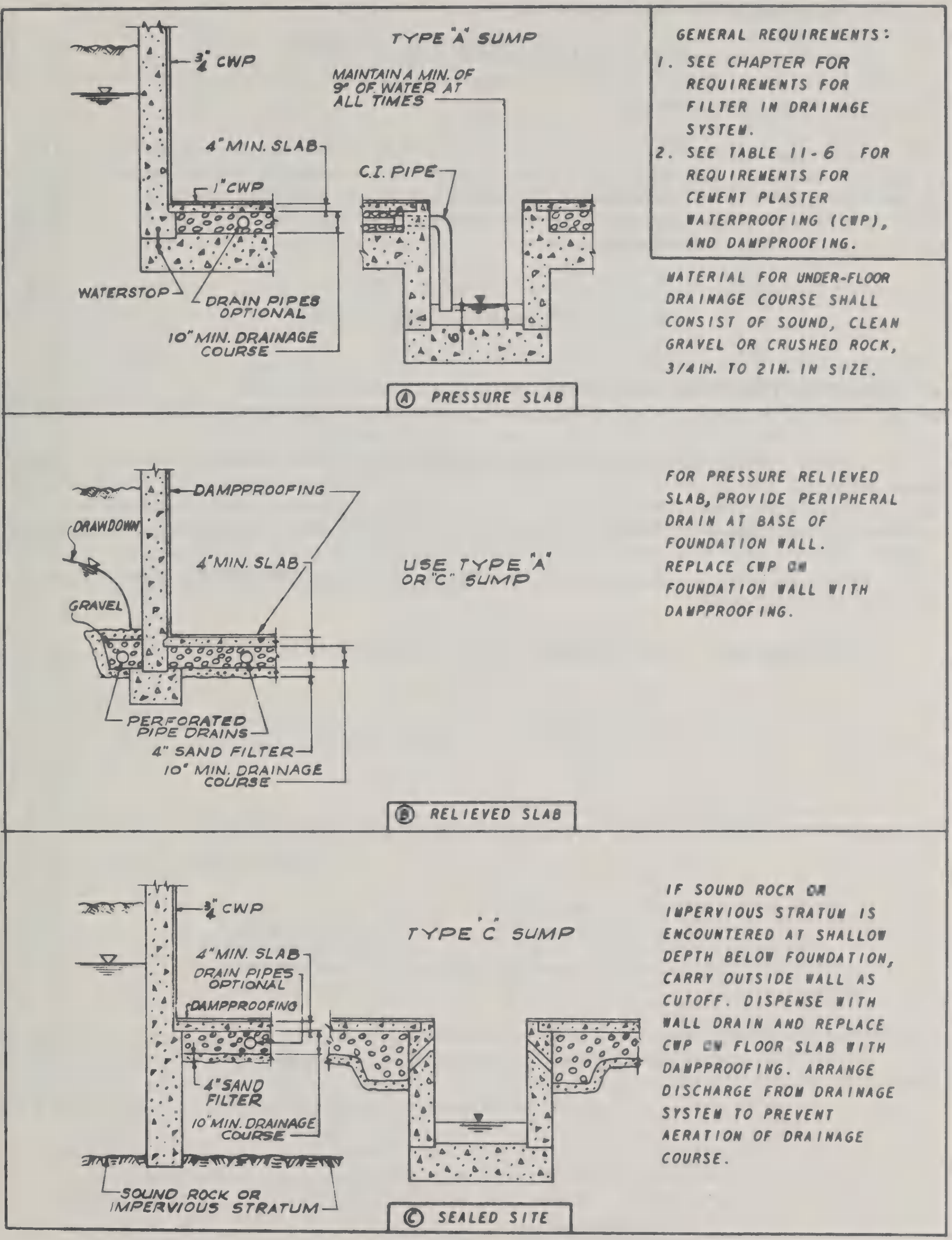


FIGURE 11-16
Typical Foundation Drainage and Waterproofing

(2) To provide a relatively stiff mat over soft subsoils to spread bearing pressures from column loads and decrease column settlements.

(3) To bridge over subsoils with erratic hard and soft spots or small cavities.

(4) To accelerate subsoil consolidation and to eliminate all or part of settlement of the completed structure when used with surcharge.

b. Compaction Control. Rigidity, strength, and homogeneity of many natural soils may be increased by controlled compaction with appropriate equipment. Where coarse grained borrow can be obtained, its use for structural fill should be evaluated to decrease cost or improve performance of spread foundations. See Table 9-4 for compaction and control requirements of structural fills. See Table 11-5 for requirements for fills beneath slabs supporting wheel loads.

Section 5. TOWER GUY ANCHORAGES

1. ANCHORING TOWER GUY LOADS. For anchoring tower guy loads, consider the deadman anchorage, the drilled-in rock anchorage, and the piling anchorage, and select on the basis of feasibility and economy.

a. Deadman Anchorages. Figure 11-12 indicates recommended safety factors and analysis for deadman anchorages. In weak soil, it may be feasible to replace a considerable volume 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.

b. Anchorages in Rock. Anchorages in rock should be designed as indicated in Figure 11-10 and Section 2, Paragraph 6a above.

c. Piling Anchorages. Piling anchorages should be designed as in Figure 13-12.

CHAPTER 12. DEEP FOUNDATIONS

Section 1. INTRODUCTION

1. **SCOPE.** Deep foundations include those types that extend a significant depth below ground surface to reach strong and incompressible strata. This chapter contains criteria for analysis and design of deep foundations and outlines construction requirements and problems of foundations on rock.

2. **RELATED CRITERIA.** See NAVFAC DM-2 for criteria for loads applied to foundations by various structures, including seismic loads, and structural design of foundations.

3. **APPLICATIONS.** Deep foundations are used to avoid weak or compressible surface strata, to provide stability against scour, dredging, or excavation of nearby surface materials, and to provide a base of great mass capable of mobilizing large passive pressures to resist overturning. Economic or security considerations may require multistory basements that involve the construction problems of deep foundations. Generally, however, footings or mats for deep basements are designed as spread foundations.

a. **Deep Foundation Types.** For principal types of deep foundations used in land construction, see Table 12-1. For deep foundations in open water, see Table 12-2. In selecting a foundation scheme, compare piles and deep foundations for cost, function, and ease of construction. Pile types and deep foundations merge in the case of cast-in-place concrete caissons.

Section 2. ALLOWABLE LOADS

1. **LIMITATIONS.** Generally, allowable loads on piers or caissons are governed by a safety factor against shear failure. Settlement limitations usually do not control the allowable load if a suitable safety factor is provided.

2. **BEARING CAPACITY.** See Figures 12-1, 12-2, and 12-3 for ultimate bearing capacity of piers and caissons, where ratio of depth of embedment D to minimum foundation width B exceeds one. The safety factor against shear failure is expressed as:

$$\text{Safety Factor} = \frac{\begin{bmatrix}\text{Ultimate bearing}\\ \text{capacity of foundation}\end{bmatrix} - \begin{bmatrix}\text{Intensity of effective surcharge of}\\ \text{material above foundation base}\end{bmatrix}}{\begin{bmatrix}\text{Total applied pressure at}\\ \text{foundation base, including effective}\\ \text{weight of foundation}\end{bmatrix} - \begin{bmatrix}\text{Intensity of effective surcharge of}\\ \text{material above foundation base}\end{bmatrix}}$$

See Figure 12-4 for examples of safety factor computations for deep foundations. Provide a safety factor between 2.5 and 3 for dead plus normal live load. Selection of a safety factor between these limits depends on uniformity of foundation conditions and thoroughness of exploration and testing programs. Provide a safety factor of 2 for dead plus normal and transient live loads.

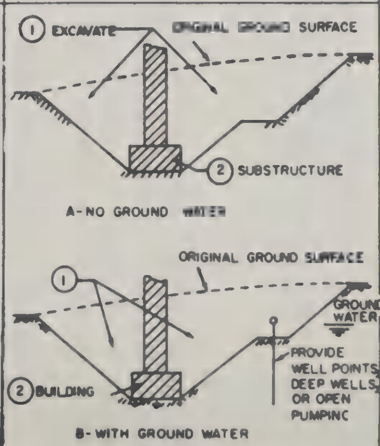
a. **Strength Assumptions.** Obtain shear strength parameters C and Φ as follows:

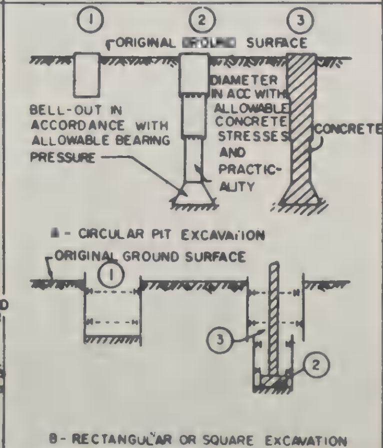
(1) For cohesionless, coarse grained soils, determine Φ from effective stress envelopes.

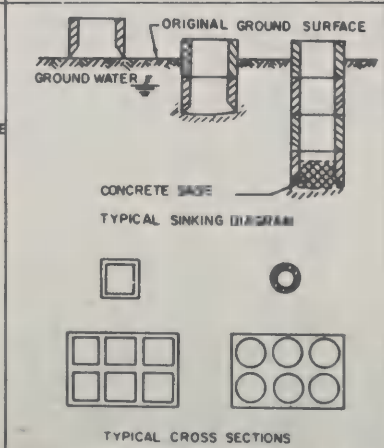
(2) For cohesive fine grained soils, determine shear strength C from U or UU triaxial tests.

TABLE 12-1
Deep Foundations on Land

	Substructure built after completion of excavation		Substructure construction concurrently with excavation	
Type	Foundation in open cut.	Substructure placed in sheeted excavation	Open caisson.	Pneumatic caisson.
Procedure	1) Excavate without sheeting and bracing at stable slopes to required subgrade level. Dewater excavation and drawdown ground water as required. 2) Complete substructure from subgrade working upward. 3) Backfill as required around the substructure.	1) Install sheeting or casing, in stages if necessary, and brace sheeting as required. 2) Excavate to required subgrade, controlling water at base of excavation. 3) Complete substructure from subgrade working upward.	Shell of foundation is sunk under its own weight or with the use of additional weights and auxiliary means to its final position while excavation is in progress within it. Foundation support is obtained by skin friction and end bearing on the concrete base placed at the bottom of the caisson.	Same procedure is applied as for open caisson except that the presence of unstable materials, difficult underseepage conditions, and the need for entry of workers requires stabilization by compressed air. Compressed air may be required for only certain parts of the sinking operation sufficient to balance the outside hydrostatic pressure.
Supplementary requirements	1) Dewatering may be by wellpoints, deep wells, or pumping from open sumps or shallow drains. Dewatering should be such as to prevent the breakout of ground water on the slopes of the excavation and to prevent boiling or detrimental underseepage at the base of the excavation. 2) Dewatering must be continued until the structure has been completed to such a stage or reached such an age that uplift on its base will not damage the substructure.	1) Dewatering by wellpoints, deep wells, or sump pumping should be such as to prevent uplift in the bottom of the excavation. In limited areas this is usually best accomplished by dewatering outside of the excavation. 2) Sheeting may consist of circular shells, sheetpiling, or soldier piles and breastboards. 3) In some cases solidification of soils below subgrade by injection processes is considered. 4) Unless tremie placement of concrete is necessary and specially provided for, concrete should be placed in a dry excavation with no seepage through the fresh concrete.	1) Pumping is performed inside the caisson for increasing or decreasing its weight as necessary, or occasionally for unwatering the base of excavation for work in the dry. 2. Lubrication is provided by a system of jets placed on the exterior walls of the caisson using water and/or air. 3) Sounding wells are provided within the framework of the caisson for testing of the bottom or determining obstructions.	1) Lubricating jets and sounding wells similar to the open caisson are utilized. 2) Airlock and pneumatic equipment are required.
Applicability	May be economical even at great depths in areas where a sufficient edge distance is available around the structure and materials are such that stable slopes may be excavated.	Most practical and usually most economical procedure in areas of limited operating space with unstable material or difficult ground water situation. Bracing system may be designed for use as permanent floor support in basement of buildings.	Method can be utilized regardless of ground water conditions. It may prove most economical for great depths and constricted working areas.	Utilize for foundations of structures extending to great depths through unstable material.
Diagram of construction sequence				







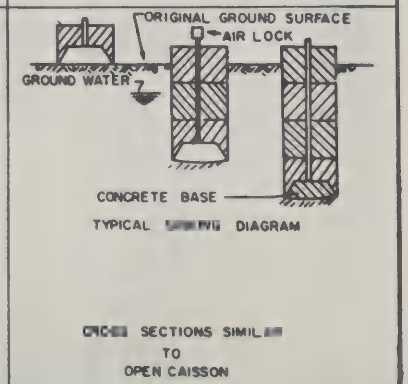
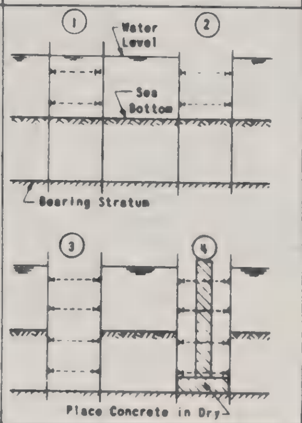
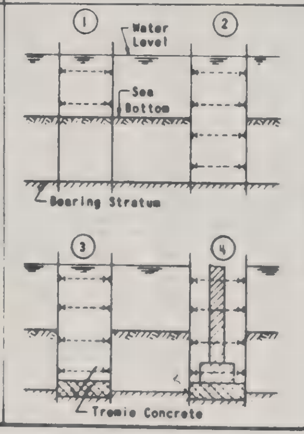
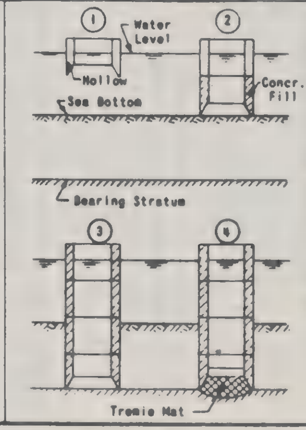
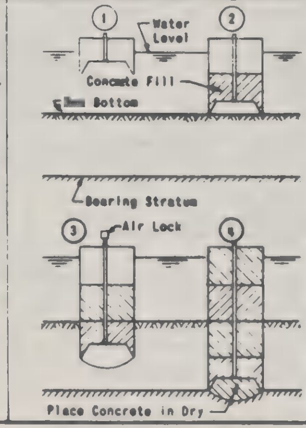
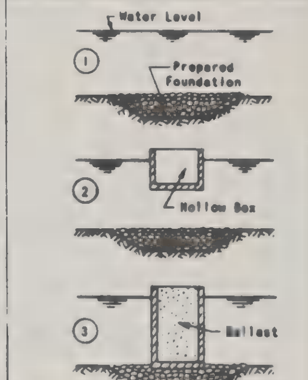
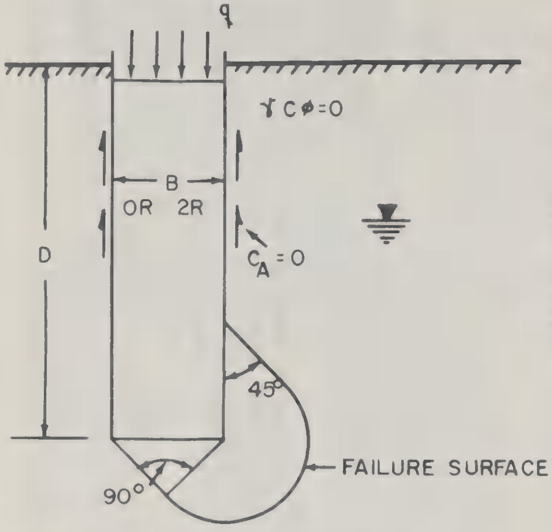
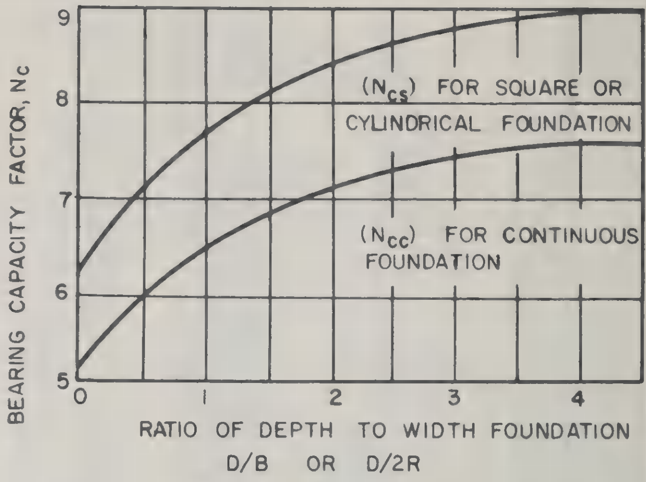


TABLE 12-2
Deep Foundations in Open Water

Type	Substructure built within an open excavation after completion of excavation		All or part of substructure built before and during progress of excavation		
	Cofferdam constructed and unwatered before erection of foundation unit	All or part of foundation construction completed before unwatering	Open caisson	Pneumatic caisson	Closed bottom floating caissons
Procedure	1) Install cofferdam and initial bracing below water to existing sea bottom. Cofferdam sheeting driven into bearing strata to control underseepage. 2) Pump down water inside cofferdam. 3) Excavate to bearing stratum completing bracing system during excavation. 4) Construct foundation within completed and unwatered cofferdam.	1) Install cofferdam and initial bracing below water to existing [illegible] bottom. 2) Excavate under water and place additional bracing to subgrade in bearing stratum. 3) Seal bottom with tremie mat of sufficient weight to balance expected hydrostatic uplift. 4) Pump out cofferdam and erect remainder of foundation structure.	1) Float caisson shell into position. 2) Build up shell in vertical lifts and place fill within shell until it settles to [illegible] bottom. 3) Continue buildup and excavate by dredging within caisson so as to sink it through unsuitable upper strata. 4) Upon reaching final elevation in bearing stratum, pour tremie base.	1) Float caisson into position. 2) Build up on top of caisson in vertical lifts until the structure settles to [illegible] bottom. 3) Continue buildup and excavate beneath the caisson, using compressed air when passing through unstable strata. 4) Pour concrete base in the dry upon reaching final position in the bearing stratum.	1) Prepare subgrade at [illegible] bottom by dredging, filling, or combination of dredging and filling. 2) Float caisson into position. 3) Sink caisson to prepared foundation at the [illegible] bottom by [illegible] of ballast.
Supplementary requirements . .	1) Guide piles or template required for driving cofferdams. 2) Cofferdam designed for high water, ice forces, or load of floating debris. 3) Cellular wall or double-wall cofferdams will eliminate or reduce required bracing system.	1), 2) and 3) same as unwatered cofferdam. 4) Relief of water pressures below tremie slab may be used to decrease the total thickness of slab required.	1) Provide anchorage or guides for caisson shells during sinking. 2) Floating and sinking operations can be facilitated by the use of false bottoms or temporary domes. 3) Dredging operations may be assisted by the use of jets or airlifts.	1) 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.	1) Provide anchorage or guides to protect floating caisson against water currents. 2) Backfill for suitable foundation should be clean granular material and may require compaction in place under water by blasting or by driving temporary displacement piles
Applicability	Procedure is used if base of excavation in bearing stratum is stable against heaving failure and if sheeting can be driven to provide cutoff in the bearing stratum. Can be utilized to depths of about 50 ft below high water.	Procedure is used where subgrade material is hard or contains boulders and penetration of cofferdam sheeting is limited; or where subgrade material is such that a satisfactory cutoff of underseepage cannot be attained by [illegible] reasonable length of sheeting. Can be used up to about 60 ft depth below high water. Completion of construction entirely below water is possible but unlikely.	Generally appropriate for depths exceeding 50 to 60 [illegible] and when final subgrade in the bearing stratum [illegible] not threatened by uplift from underlying pervious strata.	Generally required for sinking to great depths where inflow of material during excavation can [illegible] damaging to surrounding areas and/or where uplift is [illegible] threat from underlying pervious strata.	Used primarily for wharfs, piers, bulkheads, and breakwaters in water not more than 40 ft deep.
Diagram of construction procedures.					

FOUNDATION IN UNIFORM COHESIVE SOIL





ASSUMED CONDITIONS

1. SOIL IS UNIFORM TO A MINIMUM DEPTH (D) BELOW BASE OF FOUNDATION.
2. FOUNDATION HAS ROUGH BASE.
3. ADHESION ON SIDES OF FOUNDATION IS NEGLECTED.
4. VERTICAL LOAD IS CONCENTRIC.
5. SOIL HAS UNDRAINED SHEAR STRENGTH = C.

ULTIMATE BEARING CAPACITY, q_{ult}:

CONTINUOUS FOUNDATION:

$q_{ult} = CN_{cc} + \gamma D$

γD = INTENSITY OF EFFECTIVE SURCHARGE OF MATERIAL ABOVE BASE OF FOUNDATION

SQUARE FOUNDATION:

$q_{ult} = CN_{cs} + \gamma D$

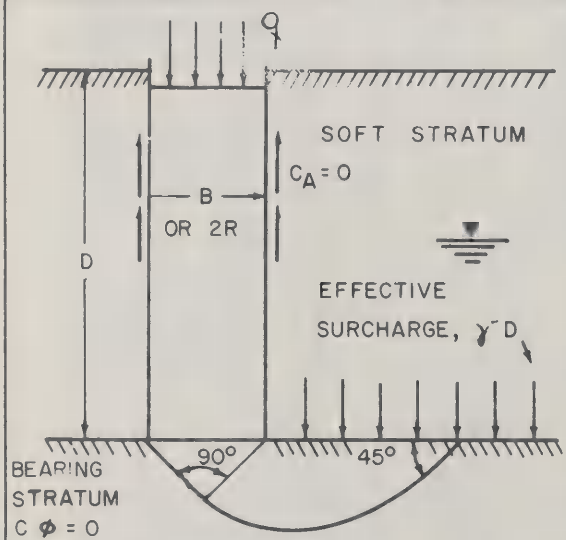
CYLINDRICAL FOUNDATION:

$q_{ult} = CN_{cs} + \gamma D$ (RADIUS $R = B/2$)

RECTANGULAR FOUNDATION:

$q_{ult} = CN_{cR} + \gamma D$ $N_{cR} = N_{cc}\left[1 + 0.2\left(\frac{B}{L}\right)\right]$

FOUNDATION IN BEARING STRATUM OVERLAIN BY SOFT MATERIAL



ASSUMED CONDITIONS SAME AS ABOVE EXCEPT THAT STRENGTH OF SOFT STRATUM IS IGNORED.

ULTIMATE BEARING CAPACITY, q_{ult}:

CONTINUOUS FOUNDATION:

$q_{ult} = CN_c + \gamma D$

SQUARE OR CYLINDRICAL FOUNDATION:

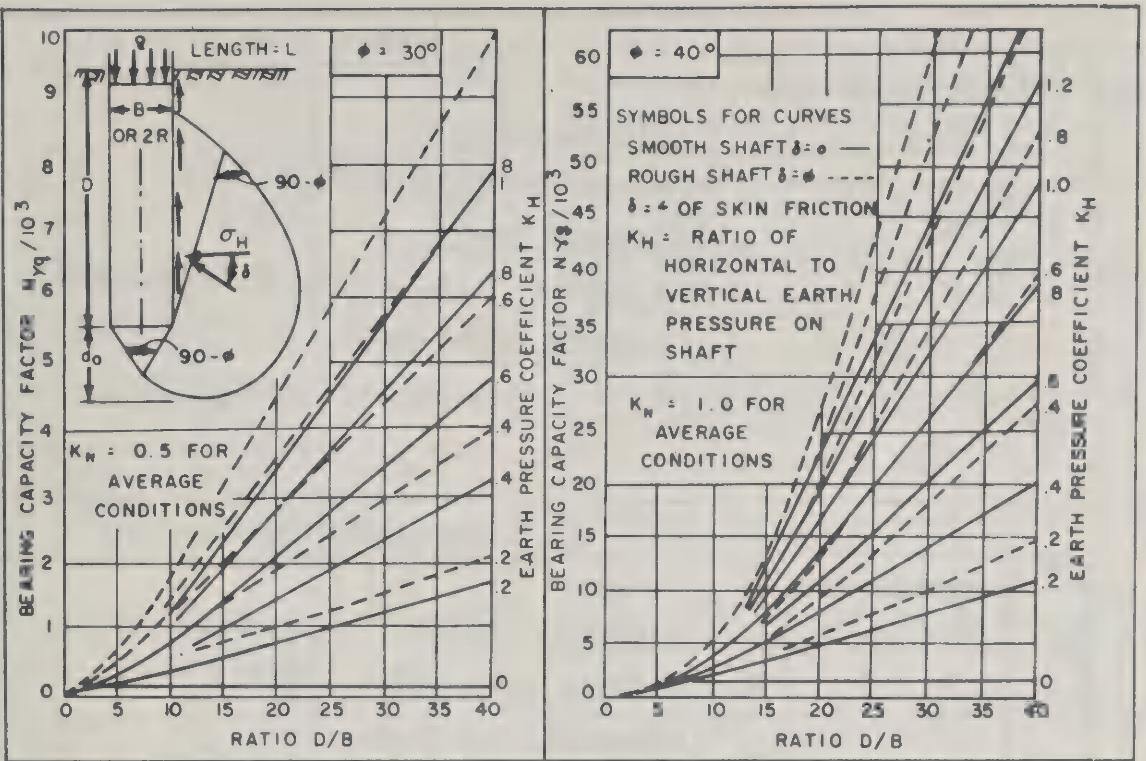
$q_{ult} = 1.3\ CN_c + \gamma D$

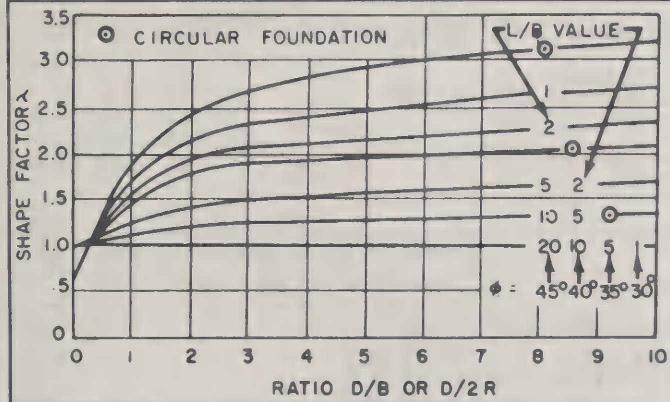
RECTANGULAR FOUNDATION:

$q_{ult} = CN_c\left[1 + 0.3\left(\frac{B}{L}\right)\right] + \gamma D$

OBTAIN VALUES OF N_c FROM FIG. 11-1.

FIGURE 12-1
Ultimate Bearing Capacity of Deep Foundations in Cohesive Soil





FOUNDATION IN UNIFORM SOIL: ULTIMATE BEARING CAPACITY FOR ALL FOUNDATION SHAPES.

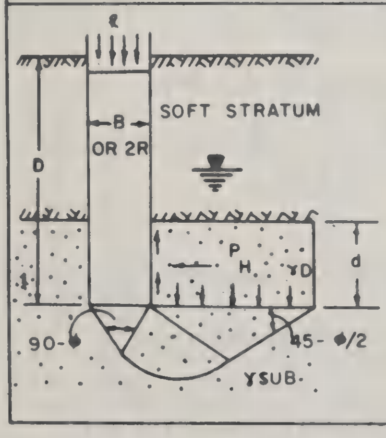
GROUND WATER AT DEPTH $d_o \geq B$

$q_{ult} = (0.5 B \gamma_T N_{\gamma q})\lambda$

FOR CYLINDRICAL FOUNDATION: 0.5B = R

FOR WATER AT GROUND SURFACE SUBSTITUTE γ_{SUB} FOR γ_T.

FOR WATER AT INTERMEDIATE LEVEL, INTERPOLATE BETWEEN THESE VALUES OF q_{ult}.



FOUNDATION IN BEARING STRATUM OVERLAIN BY SOFT MATERIAL, ULTIMATE BEARING CAPACITY:

CONTINUOUS FOUNDATION:

$$q_{ult} = \gamma D N_q + \gamma_{SUB}\left(\frac{B}{2}\right) N_\gamma + \frac{2 P_H \tan\delta}{B}$$

SQUARE OR RECTANGULAR FOUNDATION:

$$q_{ult} = \gamma D N_q + 0.4 \gamma_{SUB} B N_\gamma + \frac{2(L+B) P_H \tan\delta}{B \times L}$$

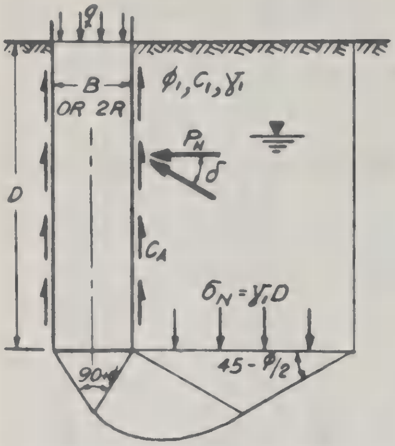
CYLINDRICAL FOUNDATION:

$$q_{ult} = \gamma D N_q + 0.6 \gamma_{SUB} R N_\gamma + \frac{2 P_H \tan\delta}{R}$$

FACTORS N_q N_γ FROM FIG. 11-1. γD = EFFECTIVE SURCHARGE ABOVE FOUNDATION BASE. P_H = RESULTANT HORIZONTAL FORCE IN HEIGHT (d) PER FOOT OF WIDTH OF FOUNDATION.

FIGURE 12-2
Ultimate Bearing Capacity of Deep Foundations in Cohesionless Soils

FOUNDATION IN UNIFORM SOIL



CONTINUOUS FOUNDATIONS

$$q_{ult} = cN_c + \gamma_1 D N_q + \gamma_1 \frac{B}{2} N_\gamma + \frac{2(C_A D + P_H \tan\delta)}{B}$$

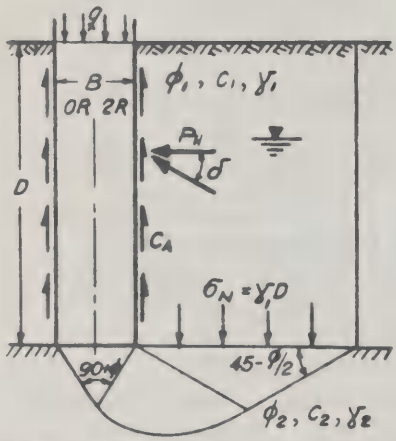
RECTANGULAR FOUNDATIONS

$$q_{ult} = cN_c(1+.3\,B/L) + \gamma_1 D N_q + 0.4\gamma_1 B N_\gamma + \frac{2(L+B)(C_A D + P_H \tan\delta)}{BL}$$

CYLINDRICAL FOUNDATIONS

$$q_{ult} = 1.3cN_c + \gamma_1 D N_q + 0.6\gamma_1 R N_\gamma + \frac{2(C_A D + P_H \tan\delta)}{R}$$

FOUNDATION IN STRATIFIED SOIL



CONTINUOUS FOUNDATIONS

$$q_{ult} = c_2 N_{c2} + \gamma_1 D N_{q2} + \gamma_2 \frac{B}{2} N_{\gamma 2} + \frac{2(C_A D + P_H \tan\delta)}{B}$$

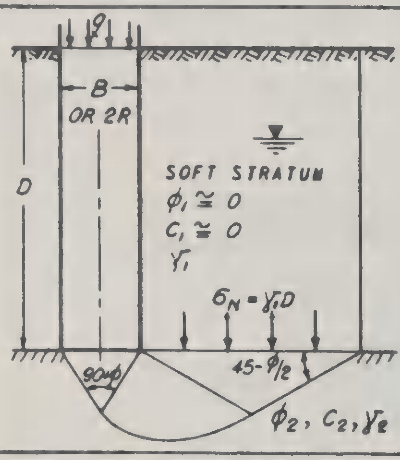
RECTANGULAR FOUNDATIONS

$$q_{ult} = c_2 N_{c2}(1+.3\,B/L) + \gamma_1 D N_{q2} + 0.4\gamma_2 B N_{\gamma 2} + \frac{2(L+B)(C_A D + P_H \tan\delta)}{BL}$$

CYLINDRICAL FOUNDATIONS

$$q_{ult} = 1.3c_2 N_{c2} + \gamma_1 D N_{q2} + 0.6\gamma_2 R N_{\gamma 2} + \frac{2(C_A D + P_H \tan\delta)}{R}$$

FOUNDATION IN BEARING STRATUM OVERLAIN BY SOFT MATERIAL



CONTINUOUS FOUNDATIONS

$$q_{ult} = c_2 N_{c2} + \gamma_1 D N_{q2} + \gamma_2 \frac{B}{2} N_{\gamma 2}$$

RECTANGULAR FOUNDATIONS

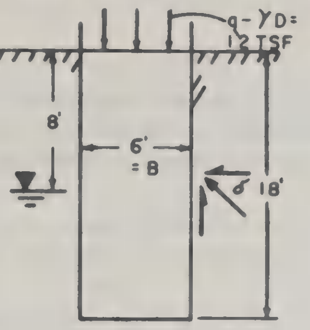
$$q_{ult} = c_2 N_{c2}(1+.3\,B/L) + \gamma_1 D N_{q2} + 0.4\gamma_2 B N_{\gamma 2}$$

CYLINDRICAL FOUNDATIONS

$$q_{ult} = 1.3c_2 N_{c2} + \gamma_1 D N_{q2} + 0.6\gamma_2 R N_{\gamma 2}$$

NOTES: N_c, N_γ, N_q = BEARING CAPACITY FACTORS FROM FIG. 11-1.
P_H = TOTAL HORIZONTAL EARTH PRESSURE PER FOOT OF FOUNDATION WIDTH.
C_A = UNIT ADHESION ON SIDES OF FOUNDATION.
L = LENGTH OF FOUNDATION IN PLAN DIMENSION.
$\gamma_1 D$ = INTENSITY OF EFFECTIVE SURCHARGE OF MATERIAL ABOVE FOUNDATION BASE.

FIGURE 12-3
Ultimate Bearing Capacity of Deep Foundations in Soils with ϕ and c



$\phi = 35^\circ$
$C = 0$
$\gamma_T = 130$ PCF
$K_H = .8$
$L = 16'$
$q - \gamma D \begin{cases} DL = 8 \text{ TSF} \\ LL = 4 \text{ TSF} \end{cases}$
$D/B = \frac{18}{6} = 3$

$N_{\gamma q}$ (FROM FIG. 12-2) = 240
λ (FROM FIG 12-2) = 1.2
K_{v1} (FROM FIG. 11-8) = 130 T/FT^3

WATER AT 1.5B = 9' BELOW BASE OF FOUNDATION:

$q_{ult} = (0.5B\gamma_T N_{\gamma q})\lambda = (0.5\times6\times.13\times240)1.2 = 112.0$ KSF

WATER AT GROUND SURFACE:

$q_{ult} = (0.5\times6\times0.068\times240)1.2 = 58.8$ KSF

WATER AT 8' BELOW GROUND SURFACE:

$q_{ult} = 58.8+(112.0-58.8)\frac{8}{18+9} = 74.5$ KSF

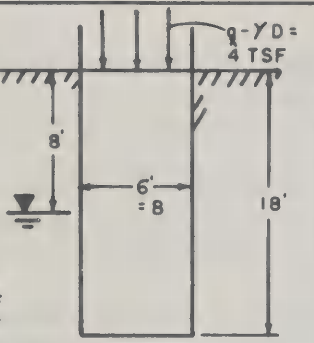
SAFETY FACTOR:

$F_s = \frac{q_{ult} - \gamma D}{q - \gamma D} = \frac{14.5 - 8\times.13 - 10\times.068}{24} = \underline{3.02}$

IMMEDIATE SETTLEMENT, $D = 3B$

$$\delta_v = \frac{4qB^2}{K_{v1}(B+1)^2}\left(1-0.5\frac{3B}{5B}\right)$$
$$= \frac{4\times12\times6^2}{130(6+1)^2}\left(1-0.5\frac{18}{30}\right)$$
$$= .189 \text{ FT.} = \underline{2.3 \text{ IN.}}$$

Ⓐ RECTANGULAR PIER IN COHESIONLESS SOIL



$C = 2$ KSF
$\phi = 0$
$C_A = 0$
$L = 16'$
$q - \gamma D \begin{cases} DL = 2.5 \text{ TSF} \\ LL = 1.5 \text{ TSF} \end{cases}$
$D/B = \frac{18}{6} = 3$

N_C (FROM FIG. 12-1) = 7.35
K_{v1} (FROM FIG. 11-8) = 100 T/FT^3

$q_{ult} = cN_c\left[1 + .2(B/L)\right] + \gamma D$

$q_{ult} = 2\times7.35\left[1+.2(6/16)\right]+.13\times8+.068\times10 = 17.5$ KSF

SAFETY FACTOR:

$F_s = \frac{q_{ult} - \gamma D}{q - \gamma D} = \frac{17.5 - 8\times.13 - 10\times.068}{8} = \underline{1.97}$

IMMEDIATE SETTLEMENT.

$$\delta_{vo} = \frac{1.05\, qBC_s\sqrt{L/B}}{K_{v1}}$$

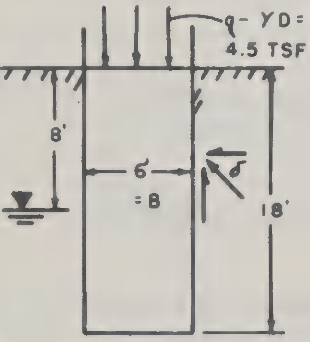
$L/B = 16/6 = 2.7$, C_s (FROM FIG 11-9) = .87 RIGID CASE

$$\delta_{vo} = \frac{1.05\times4\times6\times.78\sqrt{2.7}}{100} = 0.32 \text{ FT.}$$

$\frac{\sqrt{BL}}{D} = \frac{\sqrt{6\times16}}{18} = .55$, $\frac{\delta_{vd}}{\delta_{vo}}$ (FROM FIG. 11-9) = 0.63

$\delta_{vd} = 0.63\times.32\times12 = \underline{2.4 \text{ IN.}}$

Ⓑ RECTANGULAR PIER IN COHESIVE SOIL



$\phi = 15^\circ$
$C = 1$ KSF
$\gamma_T = 120$ PCF
$C_A = 0.$
$K_H = 0.5$
$L = 16'$
$q - \gamma D \begin{cases} DL = 3 \text{ TSF} \\ LL = 1.5 \text{ TSF} \end{cases}$
N_c (FROM FIG. 11-1) = 10.5
N_γ (FROM FIG. 11-1) = 1.1 (ROUGH)
N_q (FROM FIG. 11-1) = 3.8
$\tan\delta$ (FROM TABLE 10-1) = 0.3

$$q_{ult} = cN_c\left[1+.3(B/L)\right] + \gamma DN_q + .4\gamma BN_\gamma + 2(L+B)\frac{(C_A D + P_h \tan\delta)}{BL}$$
$$= 1.0\times10.5\left[1+.3(6/16)\right]+(.12\times8+.058\times10)3.8+.4\times.058\times6\times1.1$$
$$+ 44\frac{\left[0+(.5\times.12\frac{8^2}{2}+.5\times.12\times8\times10+.5\times.058\times\frac{10^2}{2})0.3\right]}{6\times16}$$
$$= 19.0 \text{ KSF.}$$

SAFETY FACTOR:

$F_s = \frac{q_{ult} - \gamma D}{q - \gamma D} = \frac{19.0 - .12\times8 - .058\times10}{9} = \underline{1.94}$

IMMEDIATE SETTLEMENT, ESTIMATE SETTLEMENT FROM PLATE BEARING TESTS ON PLATES OF TWO DIFFERENT SIZES, SEE FIG. 4-3 FOR METHOD OF ANALYSIS.

Ⓒ RECTANGULAR PIER IN SOIL WITH ϕ AND C

FIGURE 12-4
Examples of Computation of Safety Factor and Immediate Settlement for Deep Foundations

(3) For soils with both friction and cohesive resistance, determine strength parameters from failure envelopes for CU triaxial tests.

b. **Piers on Cohesive Soils.** See Figure 12-1 for analysis. Assume the following:

(1) Adhesion on pier sides ordinarily is neglected if upper strata are weak compared to the bearing material or if construction thoroughly disturbs upper soils. Where adhesion is included, the values are similar to those for piles (see Figure 13-1).

(2) Fluctuations of a ground water table have no effect on bearing capacity in purely cohesive soils unless shear strength is altered by ground water changes.

c. **Piers on Cohesionless Soils.** See Figure 12-2 for analysis. Assume the following:

(1) If piers are founded in coarse grained strata overlain by weak and compressible materials, ignore shear resistance acting on pier sides in the compressible stratum.

(2) Ratio of horizontal to vertical earth pressures K_H on pier sides depends on soil type and construction methods. Where construction involves outward displacement of surrounding soils, K_H ranges from 0.5 for $\Phi = 30°$, to 1.0 for $\Phi = 40°$. When construction involves inward movement of soils toward pier position, K_H may decrease to about two-thirds of these values.

d. **Soils with Friction and Cohesive Strength.** See Figure 12-3 for computation of ultimate bearing capacity for various arrangements of foundation strata.

3. UPLIFT RESISTANCE.

Safety factors against pullout are expressed as:

$$\text{Safety Factor} = \frac{\text{Effective weight of foundation} + \text{Shear resistance on foundation sides}}{\text{Uplift force}}$$

For sustained uplift, provide safety factor of 2-5. For transient uplift, provide safety factor of 2.

a. **Side Shear Resistance.** Magnitude of side shear depends on material, construction method, and foundation cross section. Where failure may occur on a shear surface through overlying backfill, values of K_H vary between at-rest and active coefficients.

b. **Shallow Foundations.** For shallow foundations or where positive contact between pier sides and soil is not certain, ignore side shear resistance. In this case, the required safety factors are reduced to between 1.5 and 2 for sustained uplift and 1 to 1.5 for transient uplift.

4. SETTLEMENTS.

See Figure 12-4 for examples of immediate settlement computation.

a. **Cohesionless Soils.** Experience indicates that settlement of a pier with depth exceeding five times its width is about one-half that of shallow footing of the same plan dimensions. For coarse grained materials whose rigidity increases with depth, compute immediate settlements due to shear strain as shown in Figure 11-8. Ignore consolidation settlements unless the bearing stratum is underlain by compressible layers. In this case compute consolidation settlements for the combined stresses of all piers in the foundation by methods of Chapter 6.

b. **Cohesive Soils.** For piers supported in stiff to hard clays, compute immediate settlements by elastic method in Figure 11-9. For piers of large dimension or groups of piers with overlapping stress patterns, add consolidation settlement for the entire foundation to immediate settlement.

Section 3. UNDERPINNING

1. **PURPOSE.** Underpinning is utilized to transfer a load carried on an existing foundation from its present bearing level to a new level at a lower depth. This operation may be necessary to prevent continuing settlement, to increase foundation load capacity, or to permit adjacent excavation without damage to existing structures.

2. **INVESTIGATIONS.** Determine by exploration the materials through which underpinning must be carried and the final bearing stratum. Where settlement of existing structures has occurred, evaluate the subsurface conditions that are responsible.

3. **PROCEDURE.** Underpinning should be performed with a carefully planned sequence of operations.

a. Load Relief. Carefully examine the structure for indications of settlement or weakness that may be accentuated during underpinning. Before excavation, reduce load on existing wall or foundation as much as possible.

b. Excavation. Limit excavation to the minimum size necessary for underpinning in stages. Sheet and brace the excavation as necessary to prevent horizontal movement of surrounding ground. Provide for dewatering as necessary for the work and to avoid piping or disturbance of bearing materials.

c. Temporary Support. Provide support of the structure over sections of the excavation by means of needles passing through, into, or under the existing structure, and supported on cribs, grillages, posts, or piles. Load bearing surfaces must be kept in close contact by the use of wedges or jacks.

d. Underpinning Members. Commence underpinning construction as soon as practicable after excavation subgrade has been exposed. Underpinning may be formed of concrete walls, piers, and caissons, or of bored piles, steel piles, or precast piles placed in sections.

(1) Foundation. Before final underpinning in concrete, the lower sections of the underpinning should be allowed to complete their set. Final contact with structure is made by wedging between steel bearing plates or by dry-pack concrete.

(2) Installing Piles. Piles are installed in sections and jacked down against a reaction provided by the existing structure. In their final position, underpinning piles are generally pretested with jacking loads of 1.5 times the intended working load. Movement under pretest load should be negligible.

4. **DESIGN.** The design of underpinning units for load carrying capacity follows procedures for deep foundations or piles. Stability and settlement of the structure and pressures on the braced excavation during construction must be evaluated. For underpinning piles the possibility of eventual removal of side friction restraint should be considered. Pretest loads must be increased to allow for side friction that may be removed with completion of adjacent work.

Section 4. CONSTRUCTION PROBLEMS

1. **STABILITY OF EXCAVATION.** Deep foundations may be constructed with stable cut slopes or may require sheeting and bracing for support of the excavation.

a. Stability of Cut Slopes. Stability of cuts in cohesionless soils depends primarily on ground water conditions beneath the slope. See Table 7-1 and 7-2 for analysis of slope stability. For temporary cuts in cohesionless soils, apply safety factor of 1.3. Unsupported cohesive slopes will stand vertically to a height between about two and three times C/γ, depending on the condition of tension cracks at top of bank.

b. Stability of Braced Cuts. If sheeting and bracing prevent horizontal movement of cut backs, determine stability of base of cut by methods of Figure 10-17, and provide safety factor of 1.5. See Figure 10-16 for analysis of sheeting and bracing for cut bank support. For cuts less than about 12 to 15 ft in depth, a cofferdam of vertical planks with wales and struts generally is practical. For greater depths, investigate steel sheet pile cofferdam or soldier piles with wood lagging, plus bracing system supported on opposite wall of trench or at excavation base. Where a cofferdam for excavation in the dry may require excessively heavy sheeting and bracing, consider the practicability of tremie construction methods. In this case, the unbalanced head of water acting upon sheeting is eliminated and a lighter cofferdam suffices. Use rotary bucket auger rigs for excavation of cylindrical holes for piers or caissons in fine grained or cohesive coarse grained soils. Depending on ground water conditions and the tendency of the soil to crack or crumble, a temporary casing may be required.

2. DEWATERING. Except where tremie construction beneath a balanced water level is utilized, provide for dewatering by methods described in Chapter 8.

a. System Required. To evaluate pumping quantities and dewatering system required, determine permeability from field pumping or variable head tests. Analyze the effect of seepage on stability or cofferdam pressures by methods of Chapter 10 as follows:

(1) Describe dewatering requirements in construction specifications.

(2) Specify piezometers or observation wells to measure drawdown water levels during construction.

(3) Specify dewatering for various construction stages in terms of drawdown required at the piezometer locations.

b. Cutoff Procedures. A peripheral cofferdam for seepage cutoff may replace or supplement dewatering. See Table 8-1 for methods to be used.

Section 5. FOUNDATIONS ON ROCK

1. ROCK TYPE. See Table 12-3 for properties of major rock types as foundation materials. For heavy structures extending to sound rock that will behave in an elastic manner, compute settlements by elastic analysis.

2. INFLUENCE OF ROCK STRUCTURE. For major foundations on rock, detailed exploration and examination of rock structure is required. For background, see Bibliography.

a. Exploration Information. Exploration should define the following:

(1) Structural characteristics such as bedding, foliation, plus strike and dip of bedding where stability problem is involved.

(2) Presence of structural defects such as joints, faults, or solution channels and cavities in sedimentary rocks.

b. Laminated Sedimentary Rock. Limestone and sandstone often occur in thin beds separated by clays or soft shales. Bearing capacity in such materials depends largely on properties of the clay or soft shales and undisturbed samples may be required for strength testing.

c. Argillaceous Rocks. Certain compaction shales, siltstones, and claystones soften or swell on exposure. Materials containing sulfates in the form of pyrites or anhydrite may hydrate with large volume expansion. Perform the following:

(1) Test cores of argillaceous sedimentary rocks for slaking by immersion in water.

(2) For materials of pronounced slaking tendency, determine swelling characteristics in consolidation tests.

TABLE 12-3
Properties of Various Rocks as Foundation Materials

Type	Rock	Typical dry unit weight, pcf	Range of modulus of elasticity, ksi	Range of compressive strength, ksi	Structural characteristics
Igneous:					
Intrusive (coarse grained).	Feldspar predominates, light colored:				
	Granite (abundant quartz)	168	4,000 to 7,000	10 to 25	Generally present as intrusion of great mass. May contain fracture system, closed except where weathering has proceeded downward from the surface. Deep localized weathering may occur at intersection of major fracture systems.
	Diorite (little quartz)	176	5,000 to 8,000		
	Iron, magnesium predominate, dark colored gabbro.	180	7,000 to 12,000	15 to 30	
Extrusive (fine grained).	Feldspar predominates, light colored:				
	Rhyolite (abundant quartz)	162	5,000 to 8,000	10 to 25	Formed in sheetlike masses characterized by extensive joint system, more open in basalt than rhyolite or andesite. May contain holes, voids, layers of volcanic ash, or pumice resulting from vulcanism concurrent with extrusion.
	Andesite (little quartz)	166	6,000 to 9,000		
	Iron, magnesium predominate, dark colored basalt.	178	7,000 to 13,000	25 to 40	
	Glassy obsidian	140	1,000 to 4,000	2 to 8	Exhibits flow structures, may be highly vesicular.
	Fragmental tuff	100	200 to 1,000	0.2 to 1	Light and relatively porous structure formed in volcanic discharge.
Metamorphic:					
Foliated (platy)	Medium grained, abundant mica schist.	167	2,000 to 5,000	5 to 15	Often intricately folded and distorted. Fracturing, softening, weathering, or deep erosion occurs in zones of intense movement. Weathering produces clayey, micaceous residue.
	Fine grained, dark colored slate	168	5,000 to 8,000	10 to 20	
Banded (imperfectly foliated).	Coarse grained, abundant quartz gneiss.	169	4,000 to 8,000	10 to 20	Less distortion than in the highly foliated rocks. Weathered residue is gritty with resistant silica particles.
Massive	Hard and brittle:				
	Quartzite (mainly quartz)	166	6,000 to 8,000	15 to 35	Quartzite and marble may be extremely hard with only fine fracture system. Some serpentines are soft to great depths.
	Marble (mainly calcite)	168	7,000 to 10,000	12 to 30	
	Relatively soft serpentine	158	1,000 to 5,000	1 to 10	
Sedimentary:					
Argillaceous (clay minerals predominate).	Fine grained, laminated:				
	Shale (clay size)	100 to 140	500 to 2,000	0.1 to 5	Wide variation in engineering properties between materials formed by compaction alone or with cementation. Compaction shales may soften, slake, and swell on exposure. Cementation types (argillite) are not sensitive to exposure.
	Siltstone (silt size)	110 to 150	500 to 2,000	0.1 to 5	
Siliceous (silica predominates).	Medium grained sandstone	147	1,000 to 3,000	4 to 12	Strength and permeability depends on type and degree of cementation. Fracturing, folding, and jointing leads to deterioration of cementing materials.
	Coarse grained:				
	Conglomerate (rounded particles)	155	1,000 to 5,000	5 to 15	
	Breccia (angular fragments)	158	1,000 to 5,000	5 to 15	

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TABLE 12-3 (Continued)
Properties of Various Rocks as Foundation Materials

Type	Rock	Typical dry unit weight, pcf	Range of modulus of elasticity, ksi	Range of compressive strength, ksi	Structural characteristics
Calcareous (calcium carbonate predominates).	Fine grained or fine crystalline:				
	Limestone (some stratified)	165	2,000 to 6,000	5 to 15	In limestone extensive cavities and solution channels may form in fracture and joint systems. Dolomite, which may be recrystallized from limestone is less soluble.
	Dolomite (some recrystallized)	167	4,000 to 8,000	7 to 20	

Note.—Values of properties given are for sound, unweathered specimens without voids or fractures, tested dry in the laboratory. Elasticity and strength depend on porosity and interlocking of grains. Elasticity and strength of specimens tested saturated, generally are 80 to 90 percent of the values shown. Values for shale and siltstone do not include materials that are essentially hard clays that may be sampled with ordinary soil sampling equipment.

(3) Slaking or swelling argillaceous rocks require special protection in excavation, similar to that provided for hard plastic clays. See Table 6-2.

d. Profile of Weathering. Generally, weathering effects diminish with depth in rock formations. In metamorphic rocks and in certain igneous masses, which are generally sound but faulted or jointed, localized weathering may extend to great depths at intersections of fault or joint systems. For major foundations on rock, review geological information and examine exposures to estimate probability of deep local weathering. For heavy concrete structures, remove weathered overburden to sound rock. Local deep excavation and dental treatment of foundation surface may be required, including slush grouting of surface irregularities.

e. Fault Systems. Fault zone gouge usually is crushed and altered mineralogically to a combination of soil and rock fragments. Construction of major projects may encounter faults active in recent geologic times. In this case, detailed evaluation of the possibility of fault movement is required. If the structure cannot be relocated, a calculated risk may have to be taken in locating the project on or near a fault zone.

3. LIMESTONE TERRAIN CONDITIONS. Special foundation problems are encountered on calcareous rocks containing solution cavities.

a. Mechanics of Solution. Ordinarily, the solution of calcium carbonate from limestones by weak carbonic acid in ground water is a slow process. It can be greatly accelerated by changes brought about by site development. For foundations above cavitated limestone, estimate by exploration the extent of cavities, the character of material residual in cavities, and whether the solution process is active or very gradual.

b. Identification of Cavities. For construction in limestone terrain, review geological data and aerial photographs for evidence of sink hole or cavity formation. Preliminary borings may indicate cavities present by sudden loss of drill water, abrupt decrease in drilling resistance, drop of casing or drill rod, or soft soil encountered within limestone. In this case, cavities should be verified by borings including core drilling with double tube core barrel. Drilling resistance must be carefully watched and the drop of drill rods correlated with missing core sections.

c. Cavities in Overburden. Cavities or solution channels deep within limestone strata are not as troublesome as cavities that cause backward erosion and collapse of surface soils. Collapse is accelerated by pumping of ground water from limestone for water supply, thus applying seepage forces directed downward from the surface; disposal of surface runoff or waste through overburden; deep excavation in overburden with concurrent drawdown of ground water; or presence of boring holes that were not sealed or grouted.

d. Remedial Measures. Consider possible location change to a more favorable site. If relocation is impractical, avoid altering the ground water regime. Consider the following possible measures against the threat of cavity formation:

(1) Grouting to provide a rigid surface layer capable of spanning cavities and resisting backward erosion.

(2) If deep solution channels are filled with soft residual materials, pile support is probably not suitable. Consider pile foundations where piles can pass through a cavitated layer and reach a sound stratum.

(3) Rigid mat or combined footing foundations with reinforced foundation walls. Where cavitation is active, design mat for loss of support over an area equal to one bay or for loss of support for any individual column.

(4) For lightly loaded structures where there is no evidence of continuing development of cavities in overburden, a compacted fill will span over small undetected voids.

(5) Eliminate leaks from utility or industrial piping and provide for inspection to avoid infiltration continuing after leaks. Collect surface runoff and convey it to a point where its infiltration will not effect structures. Avoid pumping or recharge of ground water that will increase seepage quantities.

CHAPTER 13. PILE FOUNDATIONS

Section 1. INTRODUCTION

1. **SCOPE.** This chapter concerns selection of pile type, analysis of pile capacity and design loads, and requirements for pile foundations. Field problems of pile installation and their appropriate treatment are presented.

2. **RELATED CRITERIA.** For additional criteria relating to the design of pile foundations and the selection of driving equipment and appurtenances, see the following sources:

Subject	*Source*
Design and utilization of pile driving equipment	NAVDOCKS DM-38
Details of pile splices and appurtenances	NAVDOCKS DM-38
Requirements for piles in waterfront construction	NAVDOCKS DM-25
Structural design of pile caps and pile foundations	NAVFAC DM-2

3. **APPLICATIONS.** Piles are used to transmit foundation loads to strata of adequate bearing capacity and to eliminate settlement from consolidation of overlying materials. Before specifying pile support, evaluate potential settlement and stability of competitive spread foundations. In certain situations, piles are necessary to resist lateral loads or uplift, or to provide stability against scour.

4. **INVESTIGATION PROGRAM.** Adequate subsurface exploration must precede the design of pile foundations.

a. **Information Required from Investigation.** Investigation must include the following:

(1) Geological sections showing pattern of major strata.
(2) Sufficient test data to estimate strength and compressibility of major strata.
(3) Determination of probable pile bearing stratum.

b. **Pile Tests.** For major installations, a pile testing program, including driving resistance tests, and load or pulling tests, should be planned from exploration data.

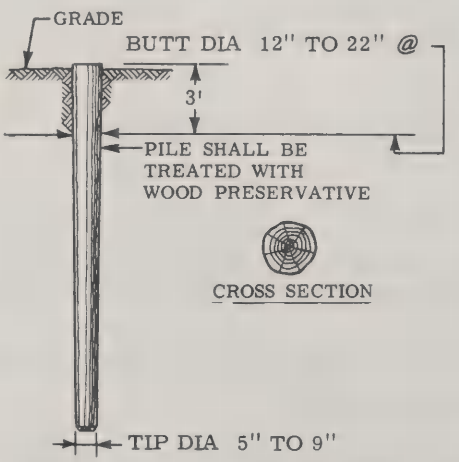
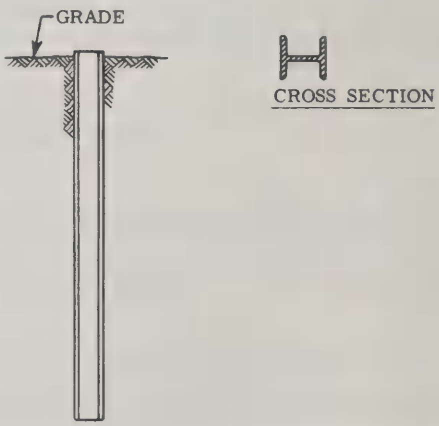
Section 2. MATERIALS AND TECHNIQUES

1. **PILE MATERIALS AND TYPES.** Table 13-1 lists nine principal pile categories of the three structural materials, wood, steel, and concrete. No exact criteria for applicability of the various pile types can be given. Selection of types should be based on factors listed in Table 13-1 and on comparative costs.

a. **Special Pile Types.** These piles consist of cylindrical columns formed of relatively low strength material compared to the three principal structural materials.

b. **Installation procedures.** Installation procedures include backfilling cylindrical holes with sand and gravel, grouting columns of soil with cement or solidifying chemical, and mechanical mixing of natural

TABLE 13-1
Design Criteria for Bearing Piles

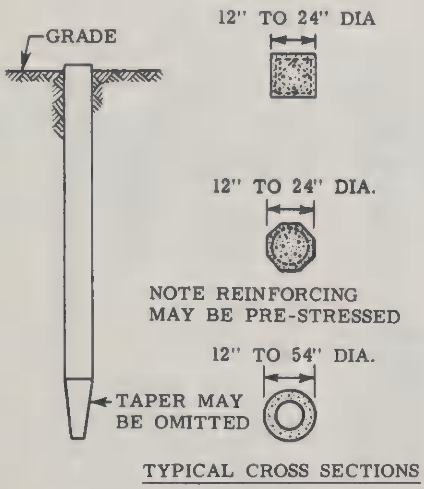
Pile type	Timber	Steel
Consider for length of....	30-60 ft	40-100 ft
Applicable material specifications.	TS-2P3	TS-P67
Maximum stresses.	Measured at most critical point, 1200 psi for Southern Pine and Douglas Fir. See U.S.D.A. Wood Handbook No. 72 for stress values of other species.	12,000 psi.
Consider for design loads of.	10-50 tons	40-120 tons.
Disadvantages.	Difficult to splice. Vulnerable to damage in hard driving. Vulnerable to decay unless treated, when piles are intermittently submerged.	Vulnerable to corrosion where exposed. BP 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. High capacity. Small displacement. Able to penetrate through light obstructions.
Remarks ...	Best suited for friction pile in granular material.	Best suited for endbearing on rock. Reduce allowable capacity for corrosive locations.
Typical illustrations.		

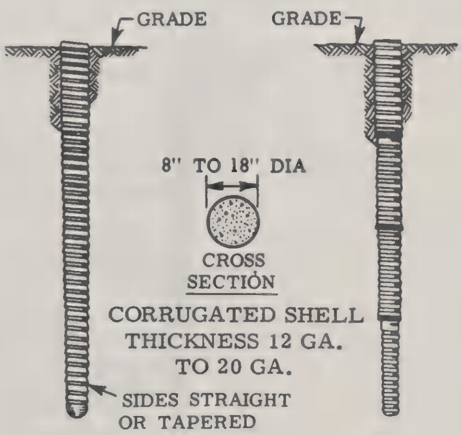
See General Notes on last page of table.

TABLE 13-1 (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.	100 ft.
Applicable material specifications.	TS-P57	ACI Code 318—For Concrete.
Maximum stresses.	For precast—33% of 28-day strength of concrete. For prestressed—$f_c = 0.33 f'_c - 0.27 f_{pe}$ (Where: f_{pe} is the effective prestress stress on the gross section).	33% of 28-day strength of concrete.
Specifically designed for a wide range of loads.	. . . .	
Disadvantages.	Unless prestressed, vulnerable to handling. High initial cost. Considerable displacement. Prestressed difficult to splice.	Difficult to splice after concreting. Redriving not recommended. Thin shell vulnerable during driving. 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.
Remarks . . .	Cylinder piles in particular are suited for bending resistance.	Best suited for medium load friction piles in granular materials.

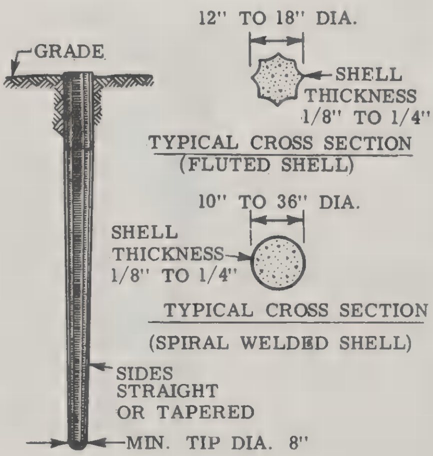
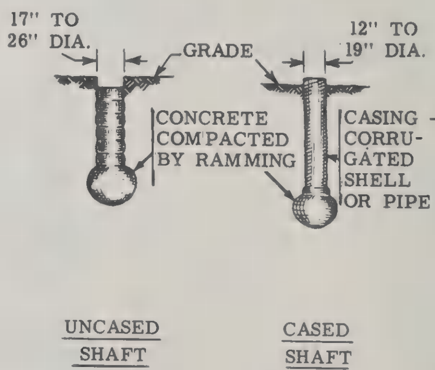
Typical illustrations.





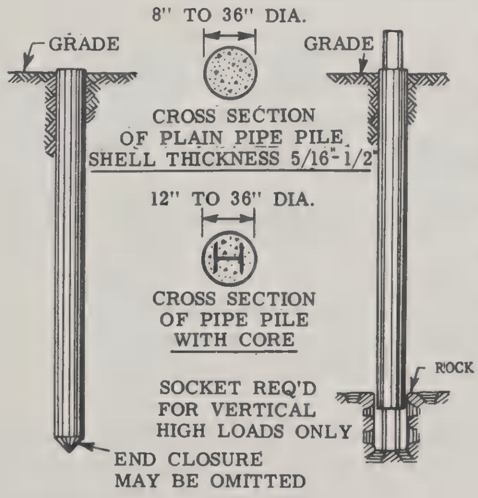
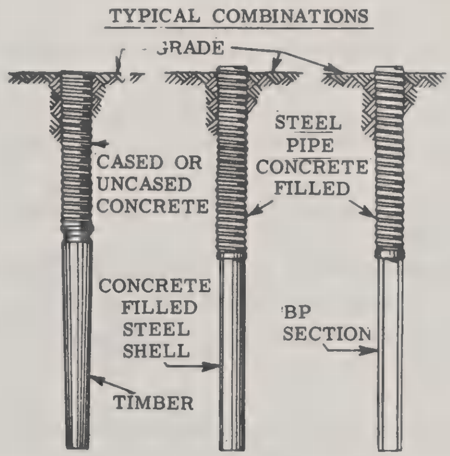
See General Notes on last page of table.

TABLE 13-1 (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 specifications.	ACI Code 318	TS-F16.
Maximum stresses.	33% of 28-day strength of concrete. 9,000 psi in shell.	33% of 28-day strength of concrete. 9,000 psi for pipe shell if thickness greater than 1/8".
Consider for design loads of. . .	50-70 tons	60-120 tons.
Disadvantages.	Hard to splice after concreting. Considerable displacement.	*Base* of footing cannot be made in clay. When clay layers must be penetrated to reach suitable material, 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. Required depths can be predicted accurately. 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. For further design requirements see Philadelphia Building Code 4-1710.
Typical illustrations.		

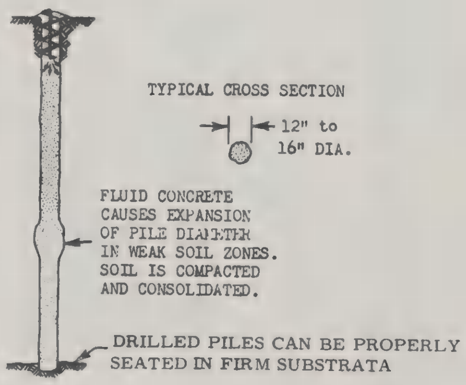
See General Notes on last page of table.

TABLE 13-1 (Continued)
Design Criteria for Bearing Piles

Pile type	Concrete filled steel pipe piles	Composite piles
Consider for length of . . .	40-120 ft	60-120 ft.
Applicable material specifications.	ASTM A7—for Core. ASTM A252—for Pipe. ACI Code 318—for Concrete.	ACI Code 318—for Concrete. ASTM-36—for Structural Section. ASTM A252—for Steel Pipe. TS-P2—for Timber.
Maximum stresses.	9,000 psi for pipe shell. 33% of 28-day strength of concrete. 12,000 psi on Steel Cores.	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 load of. . .	80-120 tons without cores. 500-1,500 tons with cores.	30-80 tons.
Disadvantages.	High initial cost. Displacement for closed end pipe.	Difficult to attain good joint between two materials.
Advantages. .	Best control during installation. No displacement for open end installation. Open end pipe best against obstructions. High load capacities. Easy to splice.	Considerable length can be provided at comparatively low cost.
Remarks . . .	Provides high bending resistance where unsupported length is loaded laterally.	The weakest of any material used shall govern allowable stresses and capacity.
Typical illustrations.		

See General Notes on last page of table.

TABLE 13-1 (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. For corrosive 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 current practice. 3. Design load capacity should be determined by soil mechanics principles limiting stresses in piles and type and function of structure. See Section 3.
Applicable material specifications.	TS-2P69.	
Maximum stresses.	33% of 28-day strength of concrete.	
Consider for design load of. . .	35-70 tons	
Disadvantages.	More than average dependence on quality workmanship. Not suitable thru peat or similar highly compressible material.	
Advantages. .	Economy. Completely nondisplacement. No driving vibration to endanger adjacent structures. High skin friction. Good contact on rock for end bearing. Convenient for low-headroom underpinning work. Visual inspection of augered material. No splicing required.	
Remarks . . .	Process patented	
Typical illustrations.		

soil in a column with cement or chemicals. Generally a considerable portion of the entire foundation area is occupied by the piles formed of stabilized material. See Table 15-4, for characteristics of mixed-in-place piles.

2. PILE DRIVING EQUIPMENT. See Table 13-2 for equipment and applications.

a. Requirements. For wood piles, minimum driving energy varies from 6,500 ft-lb for a 20-ft pile length to 15,000 ft-lb for piles over 70 ft long. Maximum driving energy for any length is 20,000 ft-lb. For concrete piles, the driving energy shall not be less than 8,700 ft-lb, except that for precast piles weighing less than 400 lb per ft, the hammer shall deliver not less than 1 ft-lb of energy per pound of pile weight. For heavier piles, driving energy shall be no less than 30,000 ft-lb.

b. Supplementary Driving Procedures. These are used for reducing driving resistance above bearing stratum, for increasing driving resistance in bearing stratum, or for advancing piles under special conditions. Where exploration or pile tests indicate need for special driving procedures, specifications should inform the contractor of these requirements and give allowable procedures.

3. SPLICES AND APPURTENANCES. See Table 13-3 for splice requirements. Uses of appurtenances such as drive shoes, reinforced tips, and lagging are given in Table 13-2.

Section 3. PILE LOAD CAPACITY

1. LIMITATIONS. The pile load capacity depends on the allowable stresses within the pile cross section and the support provided by the soil to the pile. The latter is normally the controlling factor.

a. Allowable Stresses. See Table 13-1 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 ground.

b. Soil Support. The soil must be capable of supporting the pile when it's in compression, tension and subject to lateral forces. The soil support afforded to a pile in + or −, can be computed from soil strength data, Paragraph 4 of Section 3; determined by load tests, Paragraph 3 of Section 3; and/or estimated from driving resistance, Paragraph 2 of Section 3. The approach to 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 4-4.

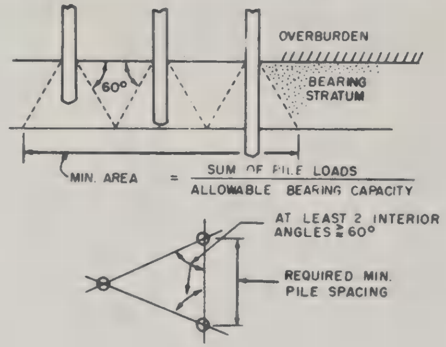
(3) *Throughout Construction Stage.* Record driving resistance of all piles for comparison with test piles and to insure against local weak subsurface formations. Record also the type and condition of cushioning material used in the pile hammer.

2. DRIVING RESISTANCE. The driving resistance in blows/inch is used to arrive at design loadings by comparing results of nearby piles in similar soil profile, and their performance in similar soil conditions.

TABLE 13-2
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 13-3
General Criteria for Installation of Pile Foundations



Item	Criteria and Limitations
Geometric requirements:	
Minimum spacing (center to center)	(1) Piles to Rock: twice the average pile diameter or 1.75 times the diagonal dimension of the 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 the 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 11-1). Piles in pile groups shall be assumed to transfer their loads to the underlying materials by spreading the load uniformly at an angle of 60 degrees 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 degree planes of adjacent piles or pile groups.
Minimum number of piles in group	Pile groups supporting superstructure loads normally consist of at least 3 piles (for arrangement see sketch), except for individual piles supporting floor slab or in cases where lateral ties are privided. Single pile support 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 ft.
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 3" 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.
Allowable loads:	
Allowable overload of piles	(1) Up to 10 percent of design load 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.
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 13-3 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 13-1 and supporting capacity of soil.
Splices	Shall be able to transmit the resultant vertical and lateral forces adequately, and, in addition shall develop no less than 50 percent of the flexural capacity of the ordinary pile cross section.
Load tests:	
Conditions requiring load test	Load tests to be performed for any of the following conditions: **(1) To verify or modify estimate of pile load capacity determined by other means.** (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.
Supervision:	
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, alinement, and penetration of pile; pile hammer used, hammer speed, and blows per foot for each foot of penetration for the full length of the pile; blows per inch for the last 6 inches of driving, except where an abrupt high increase in resistance is encountered, the final count may be reduced to penetration for the last 5 blows.

a. Driving Tests. Driving formulas may be used in this comparison as rules of thumb to supplement local experience. Because of the uncertainties of the dynamics of pile driving, the use of formulas more elaborate than those in Table 13-4 is not justified. A minimum of 3 driving tests should be made for each installation, and more if subsurface conditions are erratic.

b. Control During Construction. The embedment of piles should be controlled by specifying a minimum depth on the basis of load tests or driving tests, and in addition, requiring that the piles be driven beyond the specified depth until the driving resistance equals or exceeds the value established as necessary from the results of the test piles.

c. Formulas. Dynamic pile driving formulas should not be used as criteria for establishing load capacity without correlation with the results of soil borings and loading tests, or local experience confirmed by driving tests.

3. PILE LOAD TEST. See Chapter 4 for procedures for loading or pulling tests. See Table 13-3 for conditions requiring tests and the number of tests to be performed.

a. Planning of Tests. Precede tests with an analysis of exploration and soil test data to determine the probable bearing stratum, and to estimate the ultimate pile load. Perform a driving resistance test before loading.

b. Requirements for Tests. Load tests must simulate, as far as practicable, the conditions expected during construction. In particular, make certain that skin friction of overburden eventually to be removed does not act on test piles. If skin friction of an upper compressible stratum will act downward as drag on piles after construction, use test methods suggested in Chapter 4.

4. THEORETICAL LOAD CAPACITY. See Figures 13-1 and 13-2 for analyses of ultimate load capacity of single piles in homogeneous cohesive and cohesionless foundations.

a. Limitations. Certain factors in the analysis are poorly defined and theoretical load analysis must not be relied on exclusively but must be verified by load tests. Consider possible reversal of side friction on piles by consolidation of surrounding soil. Depending on certainty with which subsoil properties are known, apply a safety factor of 1½ to 2 maximum load. For straight tension piles use a safety factor of 2 and 2½ for tapered piles.

b. Piles in Cohesive Soils. See Figure 13-1. Experience demonstrates that pile driving permanently alters surface adhesion of clays having a shear strength greater than 500 psf. In softer clays the remolded material consolidates with time, regaining adhesion approximately equal to original strength. Shear strength for point-bearing resistance is essentially undamaged by pile driving. For analysis use adhesion tabulated in Figure 13-1 for various pile types and the original consistency of clay. 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. For slender steel piles in soft clay, compute allowable pile stresses to avoid buckling as shown in the bottom panel of Figure 13-1. Buckling for a fully embedded length of other pile types does not control pile stress.

c. Piles in Cohesionless Soils. See Figure 13-2 for formulae and theoretical load capacity of driven piles.

(1) *Compression Load Capacity.* Compression load capacity equals end-bearing capacity, as determined from theoretical bearing formulas, plus frictional capacity on perimeter surface.

(2) *Pullout Capacity.* Pullout capacity equals the frictional force on the perimeter surface above.

TABLE 13-4
Application of Pile Driving Resistance Formulas

Basic pile driving formulas (See comment in Section 3.)		
For drop hammer	For single-acting hammer	For double-acting differential hammer
$Q_{all} = \dfrac{2WH}{s+1}$	$Q_{all} = \dfrac{2WH}{s+0.1}$ {Use when driven weights are smaller than striking weights. $Q_{all} = \dfrac{2WH}{s+0.1\frac{W_D}{W_S}}$ {Use when driven weights are larger than striking weights.	$Q_{all} = \dfrac{2E}{s+0.1}$ {Use when driven weights are smaller than striking weights. $Q_{all} = \dfrac{2E}{s+0.1\frac{W_D}{W_S}}$ {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.

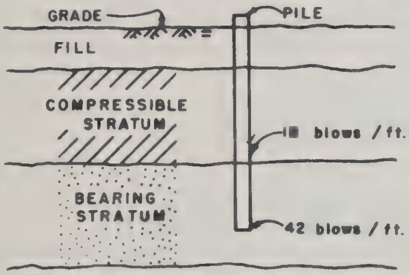
Modifications of basic pile driving formulas

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).



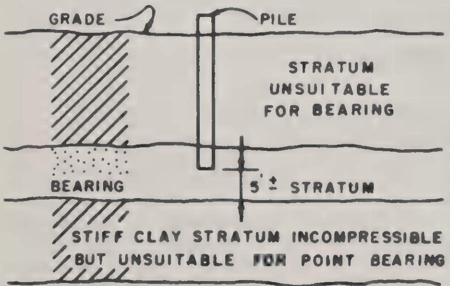
Example: required load capacity of pile Q_{all} = 25 tons
hammer energy E = 15,000 ft-lb

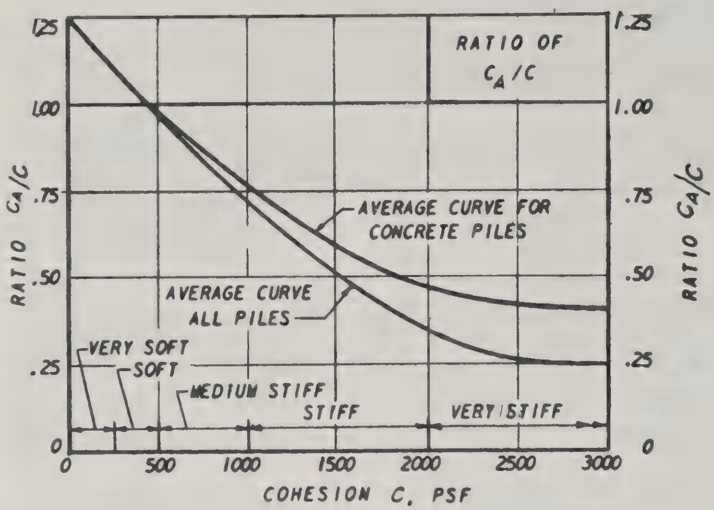
$$\frac{W_d}{W_s} < 1$$

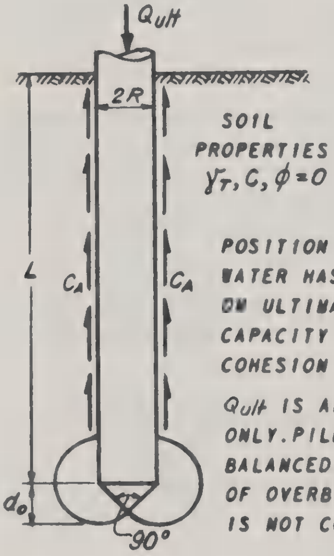
Penetration(s) as per basic formula = ½" or 2 blows per inch (24 blows/ft).

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.







POSITION OF GROUND WATER HAS NO EFFECT ON ULTIMATE LOAD CAPACITY UNLESS COHESION IS CHANGED.

Q_{ult} IS APPLIED LOAD ONLY. PILE WEIGHT IS BALANCED BY WEIGHT OF OVERBURDEN AND IS NOT CONSIDERED.

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

$$Q_{ult} = cN_{cs}\pi R^2 + C_A 2\pi RL$$

N_{cs} FROM FIG. 12-1 FOR COHESIVE SOIL.

PULLOUT CAPACITY

$$T_{ult} = C_A 2\pi RL$$

T_{ult} UNDER SUSTAINED LOAD MAY BE LIMITED BY OTHER FACTORS, SEE TEXT.

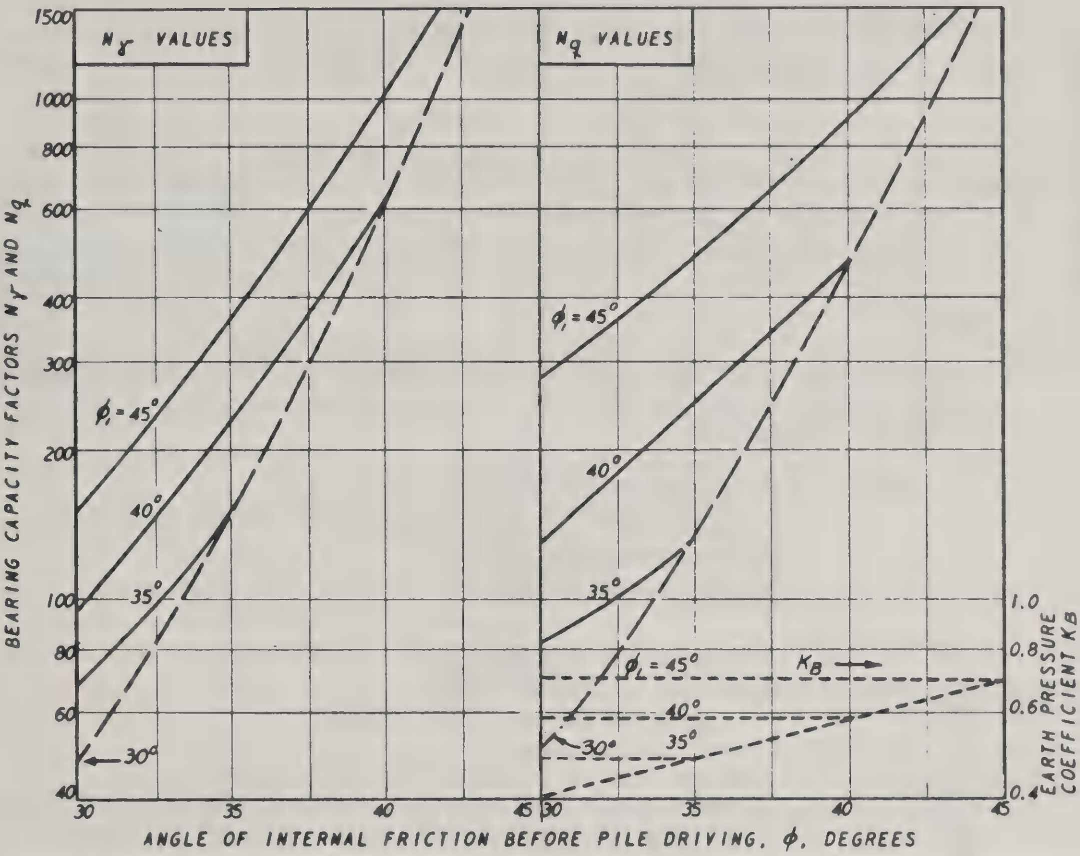
ULTIMATE LOAD FOR BUCKLING OF STEEL PILES IN SOFT CLAY.

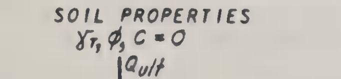
$$Q_{ult} = \lambda (C E I)^{1/2}$$

λ = 8 FOR VERY SOFT CLAY, 10 FOR SOFT CLAY (DIMENSIONLESS)
C = SOIL SHEAR STRENGTH
I = MOMENT OF ENERTIA OF CROSS SECTION
E = MODULUS OF ELASTICITY OF STEEL.
(C, I, E: POUND AND INCH UNITS)

FORMULA APPLIES TO SLENDER STEEL PILES. GENERALLY HEAVY STEEL PILES OR TIMBER OR CONCRETE PILES ARE NOT SUBJECT TO BUCKLING IF EMBEDDED IN SOIL FOR THEIR ENTIRE LENGTH.

FIGURE 13-1
Ultimate Load Capacity of Piles in Cohesive Soils





DEFINITIONS:

PILE IS DRIVEN WITHOUT JETTING OR REMOVAL OF MATERIAL WITHIN PILE.

Q_{ult} = ULTIMATE LOAD CAPACITY, APPLIED LOAD ONLY.

q = ULTIMATE BEARING CAPACITY AT TIP.

K_H = RATIO OF HORIZONTAL TO VERTICAL EARTH PRESSURE ON SIDE OF PILE ABOVE PLASTIC ZONE. AVERAGES 0.5 FOR $\phi = 30°$, 1.0 FOR $\phi = 45°$.

K_B = RATIO OF HORIZONTAL TO VERTICAL EARTH PRESSURE ON SIDE OF PILE WITHIN PLASTIC ZONE.

ϕ_1 = ANGLE OF INTERNAL FRICTION IN COMPACTED ZONE AROUND PILE TIP (4° TO 5° LARGER THAN ϕ).

$Q_{ult} = qA_S + \left(\frac{K_H}{2}\gamma_T L^2 \tan\delta\right)S$ (WATER BELOW d_o)

$q = \gamma_T R N_\gamma + K_B \gamma_T L N_q - \gamma_T L$ FOR $L \geqq 20R$

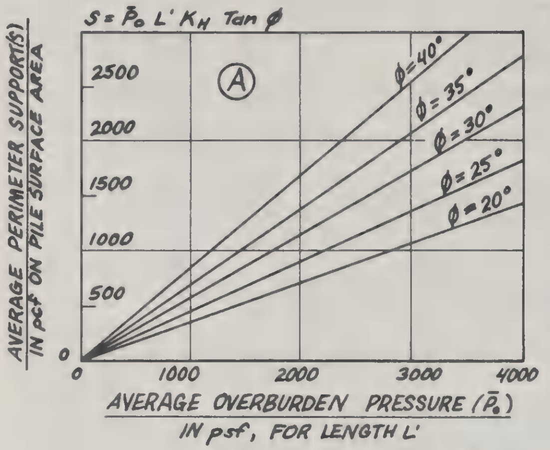
$q = \frac{L}{20R}\left(\gamma_T R N_\gamma + K_B \gamma_T L N_q\right) - \gamma_T L$ FOR $L < 20R$

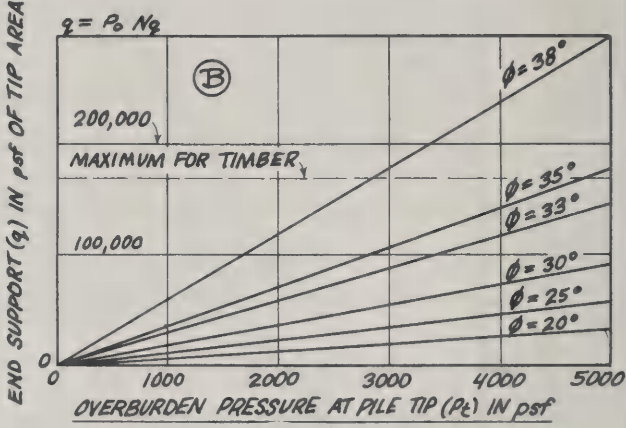
EFFECT OF WATER

WATER AT GROUND SURFACE:
SUBSTITUTE γ_{SUB} FOR γ_T IN Q_{ult}.
FOR WATER AT INTERMEDIATE LEVEL, INTERPOLATE BETWEEN LIMITING CONDITIONS.

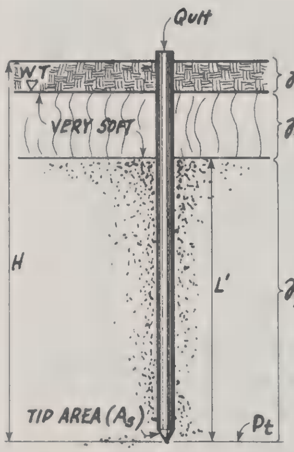
PULLOUT CAPACITY = T_{ult} = $S\left(\frac{K_H}{2}\gamma_T L^2 \tan\delta\right)$

FIGURE 13-2
Ultimate Load Capacity of Piles in Cohesionless Soil (for Alternate Method, see Figure 13-2a)





WHERE: ϕ = ANGLE OF INTERNAL FRICTION OF SOIL.
K_H = RATIO OF HORIZONTAL TO VERTICAL EARTH PRESSURE ON SIDE OF PILE. TAKEN TO BE EQUAL TO 1.0 IN GRAPH (A)
L' = LENGTH OF FIRM EMBEDMENT.
N_q = VALUES USED IN CHART (B) ARE TAKEN FROM FIGURE 11-1.



EXAMPLE SECTION

DEFINITIONS

Q_{ult} = ULTIMATE PILE LOAD CAPACITY.
= PERIMETER SUPPORT + END SUPPORT.
= S X SURFACE AREA + q X END AREA.

"S" VALUES IN GRAPH (A) ARE FOR STRAIGHT CONCRETE PILES.
FOR TAPERED PILES, DUE TO INCREASED K_H, USE 2x VALUES IN (A).
FOR STEEL OR TIMBER, DUE TO LOWER COEFFICIENT OF FRICTION, USE 0.75 X VALUES IN (A).

γ_1 ; γ_2 ; γ_3 = UNIT WEIGHTS OF SOIL, BY STRATA.
(SUBMERGED WTS BELOW WATER TABLE (WT).)

EXAMPLE PROBLEM: DETERMINE Q_{ult}

WITH: LENGTH (H) = 40'; L' = 25'; γ_3 = 65 pcf ϕ = 30°
AVERAGE DIAMETER = 10" γ_2 = 25 pcf FOR 13'
TIP AREA = 0.34 sq ft γ_1 = 80 pcf FOR 2'
TIMBER PILE (CONSIDERED TAPERED)

Q_{ult} = PERIMETER SUPPORT + END SUPPORT

PERIMETER SUPPORT:

AVERAGE OVERBURDEN PRESSURE $(\bar{P}_o) = \dfrac{(2\gamma_1 + 13\gamma_2) + (2\gamma_1 + 13\gamma_2 + 25\gamma_3)}{2}$

$\bar{P}_o$ = 1297 psf ENTER (A) AT $\bar{P}_o$ ON ϕ = 30°
S = 750 X 2 X 0.75 = 1125 psf
(TAPER) (TIMBER)
S X SURFACE AREA = 1125 psf X $\pi DL'$ = 73,000 LBS.

END SUPPORT:

OVERBURDEN PRESSURE AT PILE TIP (P_t)
$P_t = 2\gamma_1 + 13\gamma_2 + 25\gamma_3$ = 160 + 325 + 1625 = 2110 psf
ENTER (B) AT P_t = 2110 AND ϕ = 30°
THEN q = 38,000 psf
$q \times A_s$ = 38,000 psf x 0.34 sq ft = 12,900 LBS.

DESIGN LOAD = $\dfrac{Q_{ult}}{\text{SAFETY FACTOR*}}$

FOR CAPACITY AS A TENSION PILE, USE STRAIGHT PILES AND USE PERIMETER SUPPORT ONLY IN FIRM EMBEDMENT.

$\boxed{Q_{ult} = 73{,}000 \text{ LBS} + 12{,}9000 \text{ LBS} = 86^K}$

*SEE SECTION 3, PARAGRAPH 4 FOR SAFETY FACTOR.

(3) *Ultimate Capacity.* The ultimate capacity of point-bearing piles in sand may be estimated by various cone penetration test methods. See Van der Veen and Boersma, *The Bearing Capacity of a Pile Predetermined by a Cone Penetration Test* (Bibliography), for a description of one method. Add side friction resistance, where present, to point bearing capacity as shown in Figure 13-2.

d. Piles in Soils Having Both Significant Adhesion and Friction. Compute perimeter and end support for the adhesion and repeat procedure for the friction condition. For load capacity, use the condition that gives most capacity. Where pile penetrates several different strata, add supporting capacity of the individual layers, except where a soft layer may consolidate and relieve load or cause drag on the pile. No adhesion should be considered effective to the depth that jetting is conducted.

5. LATERAL LOAD CAPACITY. A pile loaded by thrust and moment at its top resists the load by deflecting to mobilize reaction of the surrounding soil. Resisting pressures applied by soil to a pile depend on relative stiffness of the pile and soil.

a. Loading Conditions. Three principal loading conditions are illustrated with the design procedures in Figure 13-3, using the influence diagrams of Figure 13-4, 13-5, and 13-6. Loading may be limited by allowable deflection of pile top or by pile stresses.

(1) *Case I.* Pile with flexible cap or hinged end condition. Thrust and moment are applied at pile top, which is free to rotate. Obtain total deflection, moment, and shear in the pile by algebraic sum of the effects of thrust and moment, given in Figure 13-4.

(2) *Case II.* Pile with rigid cap fixed against rotation at ground surface. Thrust is applied at the pile top, which must maintain a vertical tangent. Obtain deflection and moment from influence values of Figure 13-5.

(3) *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 13-6 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 13-4.

b. Soil Properties. These solutions assume a soil modulus of elasticity increasing linearly with depth as for coarse grained material and soft to medium stiff fine grained soils. For approximate values of constant of proportionality f of soil modulus with depth, see Figure 11-8. For preconsolidated stiff to hard clays, the modulus of elasticity is approximately constant with depth. Convert this constant modulus to an equivalent value that varies linearly with depth to utilize the influence values of Figure 13-4 to 13-6. See Figure 11-9 for this conversion.

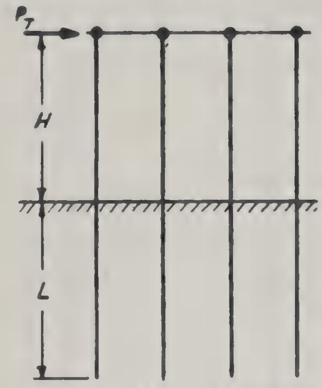
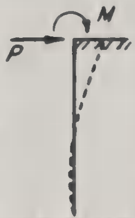
Section 4. PILE CLUSTER CAPACITY

1. ULTIMATE LOAD. For analysis of ultimate load in homogeneous subsoils, see Figure 13-7; for stratified soils, see Figure 13-8. See Figure 13-9 for examples of computation. Ordinarily, the dead weight of piles plus soil enclosed within the pile group is assumed to be balanced by the weight of soil surrounding the pile group and is ignored in the computation. To determine allowable load, apply a safety factor of 3 to the ultimate capacity of the group for dead plus normal live load and a safety factor of 2-1/2 for dead plus maximum live load.

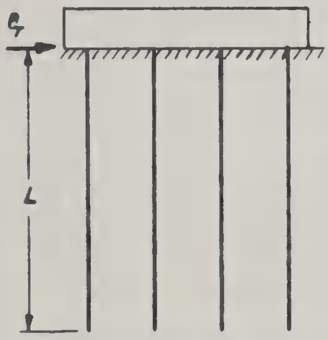
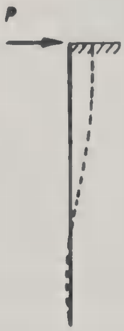
a. Group Efficiency. The efficiency of a pile group equals the ratio of ultimate capacity of pile in cluster to its capacity as an individual pile and depends on the degree of overlap of pressures transmitted by adjacent piles.

(1) *Cohesive Soils.* For cohesive soils, group efficiency increases with pile spacing as shown in Figure 13-7.

CASE I. PILES WITH FLEXIBLE CAP OR HINGED END CONDITION

CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
 n = NUMBER OF PILES	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$  DEFLECTED POSITION	FOR DEFINITION OF PARAMETERS SEE FIG. 13-5. 1. COMPUTE RELATIVE STIFFNESS FACTOR $T = \left(\frac{EI}{f}\right)^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIG. 13-4. 3. OBTAIN COEFFICIENTS F_δ F_M F_V AT DEPTHS DESIRED 4. COMPUTE DEFLECTION, MOMENT AND SHEAR AT DESIRED DEPTHS USING FORMULAS OF FIG. 13-4. NOTE: "f" values from Fig. 11-8 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 FIG. 13-5. 3. MAXIMUM SHEAR OCCURS AT TOP OF PILE AND EQUALS $P = \frac{P_T}{n}$ IN EACH PILE.

CASE III. RIGID CAP, ELEVATED POSITION

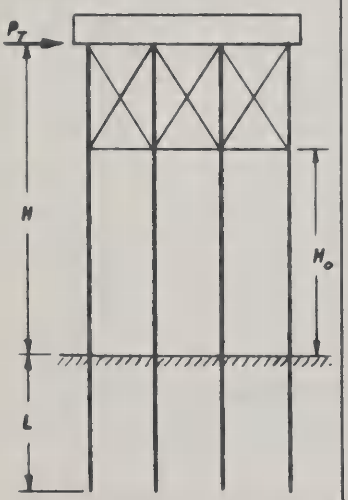
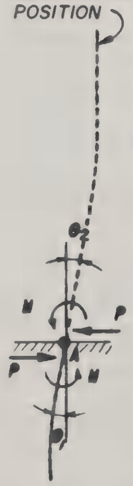
	DEFLECTED POSITION 	1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIG. 13-6 AS FOLLOWS: $\theta_1 = F_\theta\left(\frac{PT^2}{EI}\right) + F_\theta\left(\frac{MT}{EI}\right)$ 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 13-3
Design Procedure for Laterally Loaded Piles

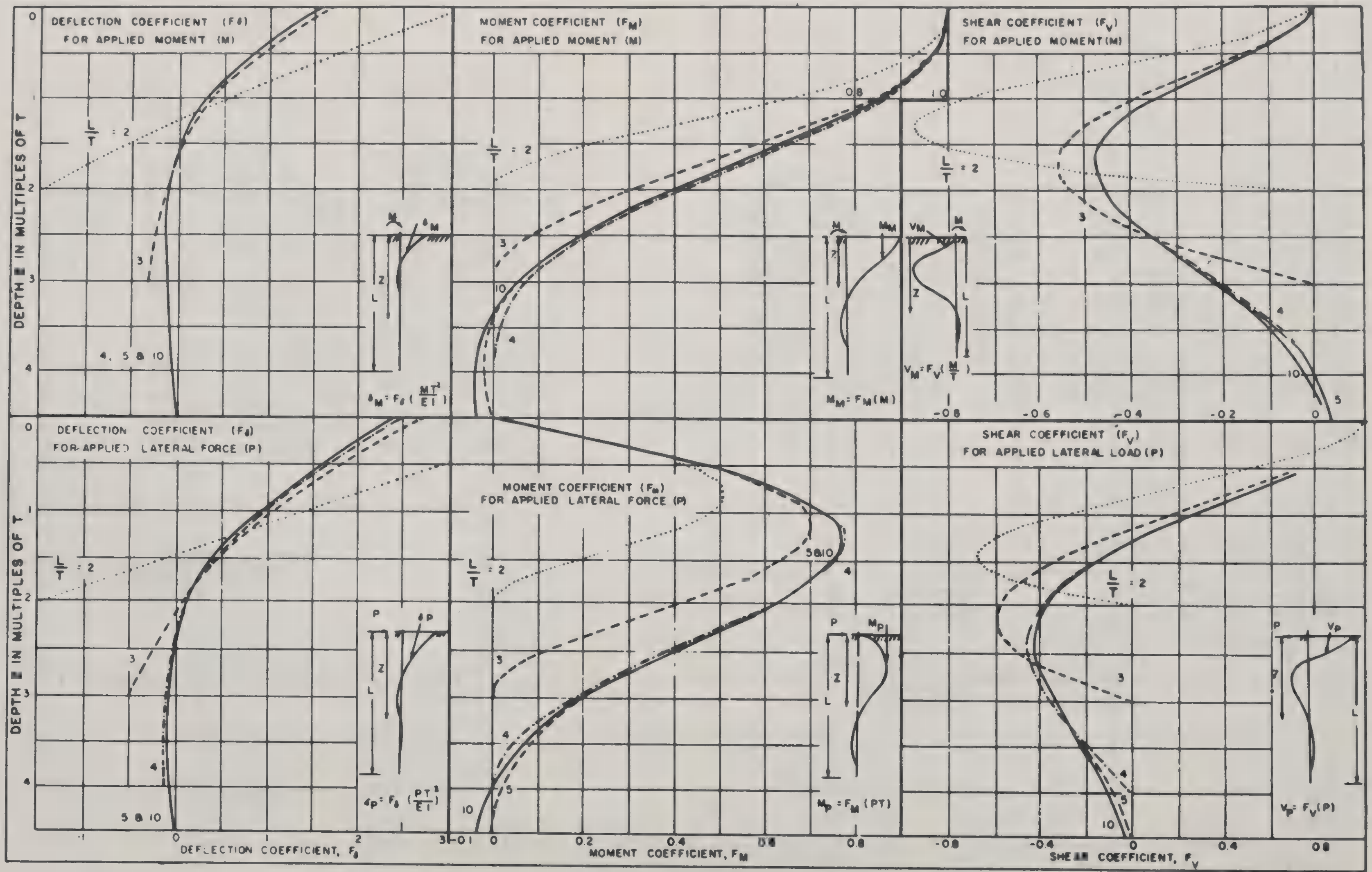


FIGURE 13-4
Influence Values for Pile with Applied Lateral Load and Movement
(Case I. Flexible Cap or Hinged End Condition)

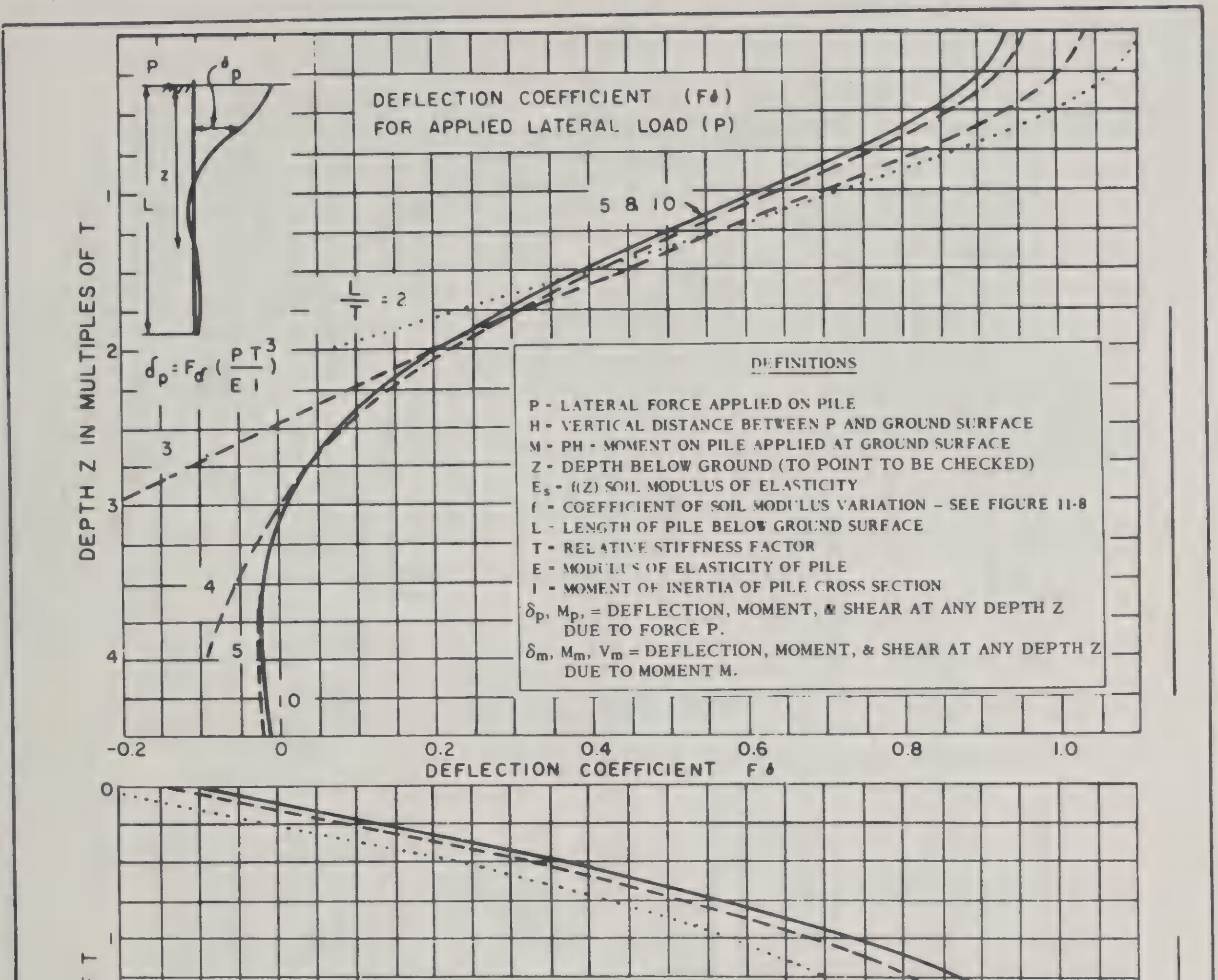
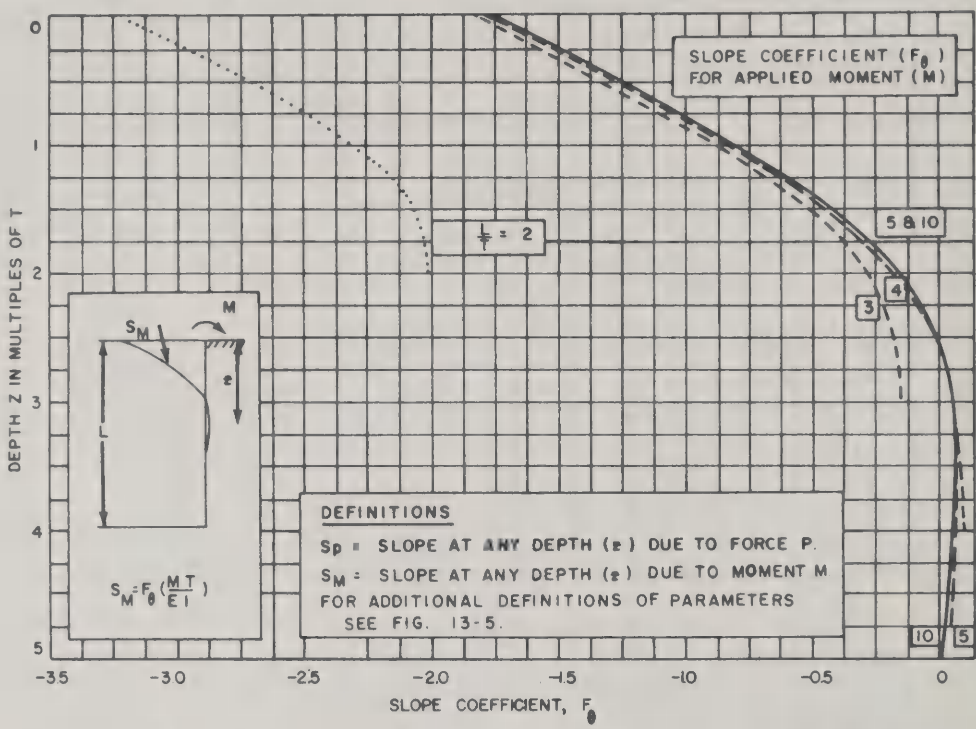


FIGURE 13-5
Influence Values for Laterally Loaded Pile
(Case II. Fixed Against Rotation at Ground Surface)



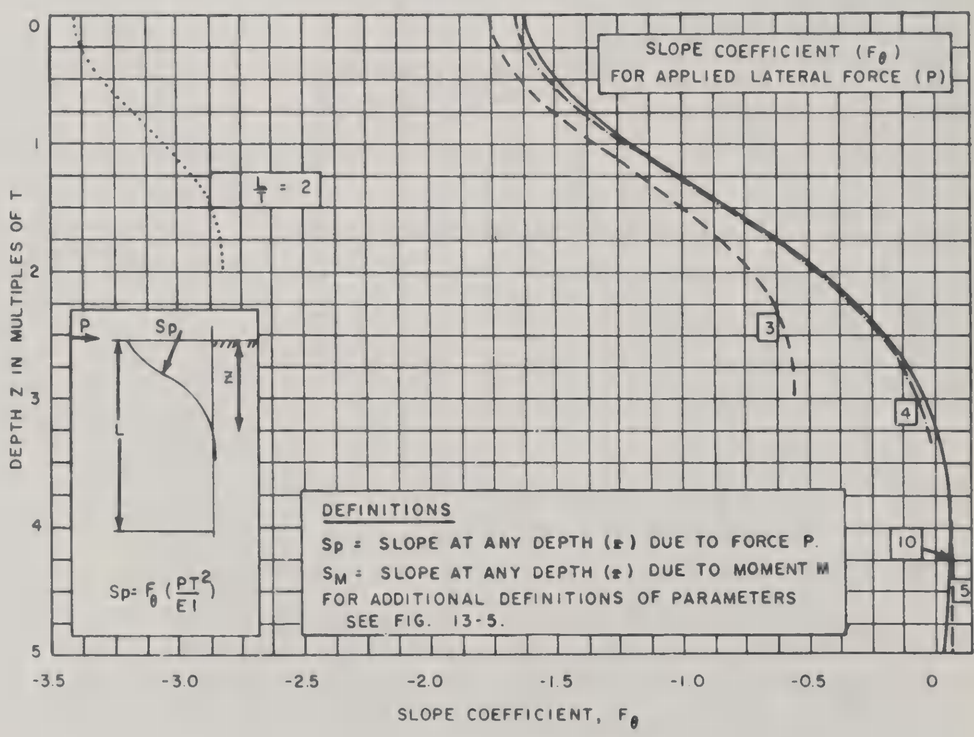
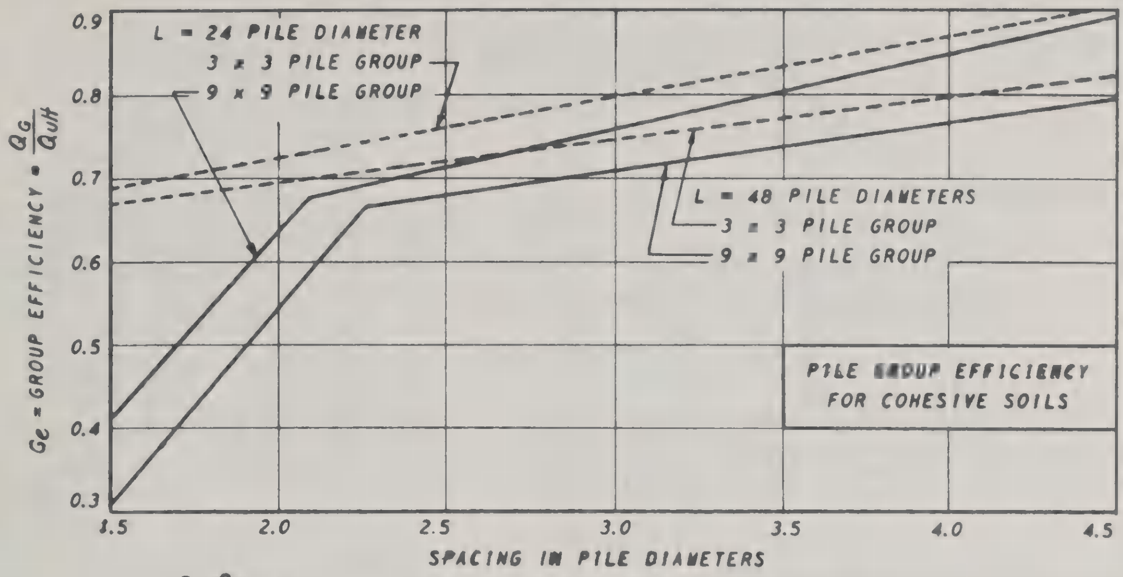
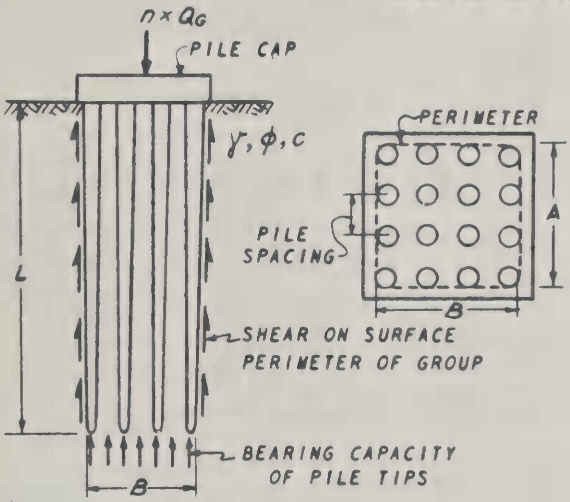


FIGURE 13-6
Slope Coefficient for Pile with Lateral Load or Moment





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.

PILES IN COHESIVE SOILS

$Q_{ult} = (cN_c)\pi R^2 + C_A 2\pi RL$ (OBTAIN C_A FROM FIG. 13-1, OBTAIN N_c FROM FIG. 12-1)

ULTIMATE LOAD OF GROUP = $nQ_G = G_e n Q_{ult}$

PILES IN COHESIONLESS SOILS $\quad A_s = B \times A \qquad P = 2(A \times B)$

PILE SPACING $\leq$ 6R: $nQ_G = Q_{ult}$ $\quad Q_{ult}$ FROM FIG. 13-2 WITH R = B/2

PILE SPACING > 16R: $nQ_G = nQ_{ult}$ $\quad Q_{ult}$ FROM FIG. 13-2 WITH R = PILE RADIUS.

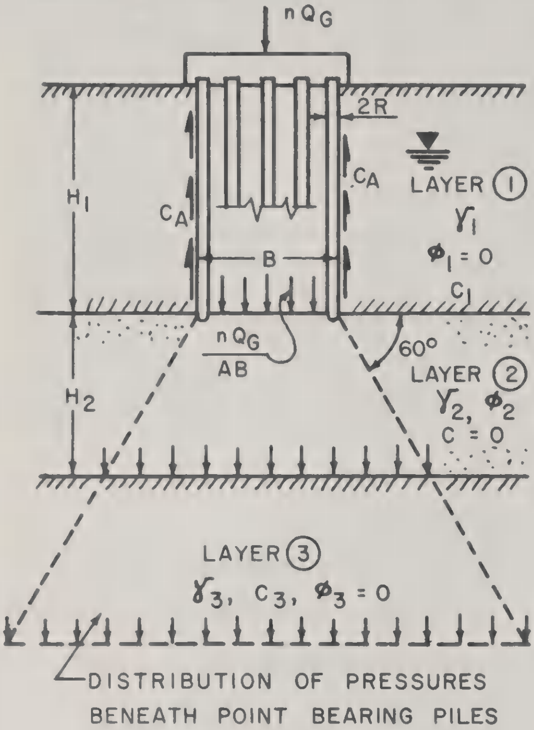
FOR PILE SPACING BETWEEN 6R AND 16R, INTERPOLATE BETWEEN THESE LIMITING CONDITIONS. FORMULAS APPLY FOR A = B $\leqq$ 2A.

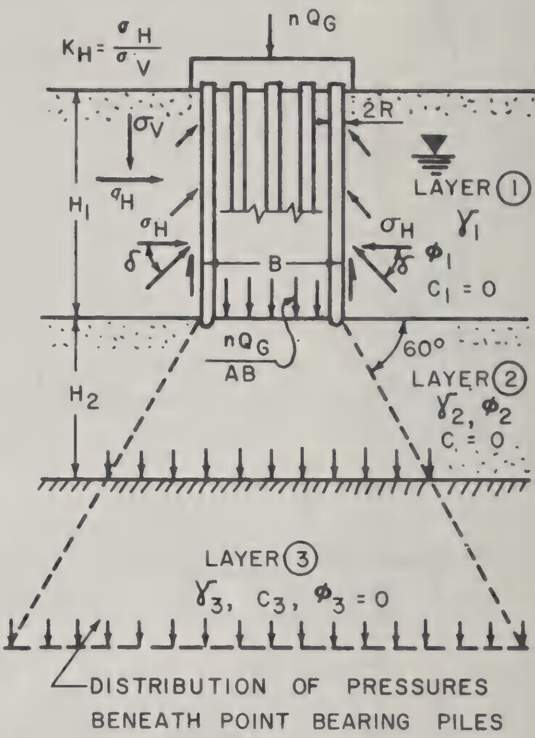
PULLOUT CAPACITY:

FOR COHESIVE OR COHESIONLESS SOILS, PULLOUT CAPACITY EQUALS TOTAL SHEAR FORCE ON SURFACE PERIMETER OF GROUP.

FIGURE 13-7
Ultimate Load Capacity of Pile Groups

OUTSIDE DIMENSIONS OF PILE GROUP IN PLAN = A × B, (B) IS SMALLER DIMENSION. PILES STOP IN TOP OF COARSE GRAINED LAYER (2). LAYER (2) IS UNDERLAIN BY COHESIVE STRATUM. LAYER (3). n = NUMBER OF PILES.





LAYER (1) IS COHESIVE ($\phi = 0$)

nQ_G = ULTIMATE LOAD CAPACITY OF GROUP

Q_{ult} = ULTIMATE CAPACITY OF SINGLE PILE (WEIGHT OF PILES NEED NOT BE INCLUDED IN APPLIED LOAD).

FAILURE IN LAYER (2) ($H_2 \geq B$)

PILE SPACING $\leq$ 6R:

$$nQ_G = (\gamma_1 H_1 N_{q2} + 0.4\gamma_2 B N_{\gamma 2}) A \times B + 2C_A (A+B) H_1 - AB\gamma_1 H_1$$

PILE SPACING > 16R: $nQ_G = n(Q_{ult})$

$$Q_{ult} = (\gamma_1 H_1 N_{q2} + 0.4\gamma_2 B N_{\gamma 2}) \pi R^2 + 2C_A \pi R H_1 - \pi R^2 \gamma_1 H_1$$

LAYER (1) IS COHESIONLESS (C = 0)

FAILURE IN LAYER (2) ($H_2 \geq B$)

IF GROUND WATER IS AT DEPTH GREATER THAN (B) BELOW TOP OF LAYER (2):

IF LAYER (1) IS ESSENTIALLY SIMILAR TO TO LAYER (2), OBTAIN nQ_G FROM FIG. 13-2.

IF ϕ_1 DIFFERS GREATLY FROM ϕ_2:

PILE SPACING < 6R:

$$nQ_G = (\gamma_1 H_1 N_{q2} + 0.4\gamma_2 B N_{\gamma 2}) A \times B + (A+B) K_H \gamma_1 \tan\delta_1 H_1^2 - AB\gamma_1 H_1$$

PILE SPACING > 16R: $nQ = n(Q_{ult})$

$$Q_{ult} = (\gamma_1 H_1 N_{q2} + 0.4\gamma_2 B N_{\gamma 2}) \pi R^2 + \pi R \gamma_1 K_H \tan\delta_1 H_1^2 - \pi R^2 \gamma_1 H_1$$

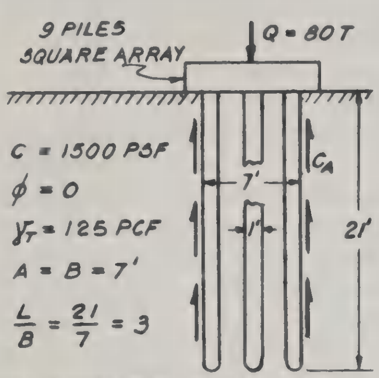
FOR PILE SPACING BETWEEN 6R AND 16R, INTERPOLATE BETWEEN THE VALUES FOR 6R AND 16R.

FOR WATER NEAR TO THE GROUND SURFACE, SUBSTITUTE $\gamma_{1\,sub}$ FOR γ_1 AND $\gamma_{2\,sub}$ FOR γ_2 IN THE ABOVE FORMULAS. INTERPOLATE BETWEEN THESE LIMITS FOR INTERMEDIATE WATER LEVEL.

IN ANY CASE THE POSSIBILITY OF FAILURE IN CLAY LAYER (3) MUST BE INVESTIGATED. THIS IS PARTICULARLY IMPORTANT IF LAYER (2) IS THIN COMPARED TO DIMENSION (B). FAILURE OF LAYER (3) OCCURS IF LOAD DISTRIBUTED ON TOP OF LAYER (3) AS SHOWN EXCEEDS $1.3C_3N_c$.

FACTORS N_c N_γ & N_q OBTAINED FROM FIG. 11-1 FOR ALL CONDITIONS EXCEPT FOR COHESIONLESS SOILS WHEN LAYER (1) IS SIMILAR TO LAYER (2), IN THIS CASE USE N_c, N_γ AND N_q FROM FIG. 13-2.

FIGURE 13-8
Ultimate Load Capacity of Pile Groups in Layered Subsoils



$C = 1500$ PSF

$\phi = 0$

$\gamma_T = 125$ PCF

$A = B = 7'$

$\frac{L}{B} = \frac{21}{7} = 3$

N_C (FROM FIG. 12-1) = 8.85

C_A (FROM FIG. 13-1) = 850 PSF

ULTIMATE LOAD CAPACITY OF A SINGLE PILE (FROM FIG. 13-1):

$$Q_{ult} = (CN_C)\pi R^2 + C_A 2\pi R L$$
$$= (1.5\times 8.85)3.14\times 0.5^2 + .85\times 2\times 3.14\times 0.5\times 21$$
$$= 66.5 \text{ KIP.}$$

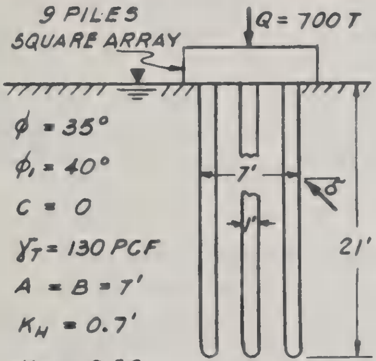
ULTIMATE LOAD CAPACITY OF GROUP:

PILE SPACING = 3 DIA., G_e (FROM FIG. 13-7) = 0.8

$$nQ_G = G_e\, n Q_{ult} = 0.8\times 9\times 66.5 = 480 \text{ KIP.}$$

$$\underline{\text{SAFETY FACTOR}} = \frac{nQ_G}{Q} = \frac{480}{160} = 3.0$$

Ⓐ PILE GROUP IN COHESIVE SOIL



$\phi = 35°$

$\phi_1 = 40°$

$C = 0$

$\gamma_T = 130$ PCF

$A = B = 7'$

$K_H = 0.7'$

$K_B = 0.59$

N_γ (FROM FIG. 13-2) = 235

N_q (FROM FIG. 13-2) = 255

$\tan\delta = \tan 0.7\phi \cong .4$

PILE SPACING = 6R, $nQ_G = Q_{ult}$ (FROM FIG. 13-7)

$$Q_{ult} = q A_3 + \left(\frac{K_H}{2}\gamma_{sub} L^2 \tan\delta\right) P$$

$$R = B/2;\ L < 20R < 10B < 70$$

$$q = \frac{L}{20R}\left(\gamma_{sub} R N_\gamma + K_B \gamma_{sub} L N_q\right) - \gamma_{sub} L$$

TAKE $R = B/2$ $\quad A_3 = B\times A \quad P = 2(A+B)$

$$= \frac{L}{10B}\left(\gamma_{sub}\times\frac{BN_\gamma}{2} + K_B\gamma_{sub} L N_q\right) - \gamma_{sub} L$$

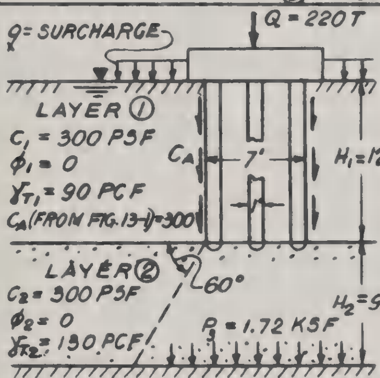
$$= \frac{21}{10\times 7}\left(.068\times\frac{7\times 235}{2} + 0.59\times .068\times 21\times 255\right) - 0.068\times 21$$

$$= 81.5 \text{ KSF.}$$

$$Q_{ult} = 81.5\times 7\times 7 + \left(\frac{0.7}{2}\times 0.68\times 21^2\times 0.4\right)(7\times 7)2 = 4{,}118 \text{ KIP}$$

$$\underline{\text{SAFETY FACTOR}} = \frac{nQ_G}{Q} = \frac{Q_{ult}}{Q} = \frac{4{,}118}{1{,}400} = 3.0$$

Ⓑ PILE GROUP IN COHESIONLESS SOIL



LAYER ③

$C_3 = 600$ PSF

$\phi_3 = 0$

N_{q2} (FROM FIG. 11-1) = 34

$N_{\gamma 2}$ (FROM FIG. 11-1) = 40

PILE SPACING = 6R, $\quad H_2 > B$:

$$Q_{ult} \text{ (FROM FIG. 13-8)} = \left(\gamma_{sub_1} H_1 N_{q2} + .4\,\gamma_{sub_2} B N_{\gamma 2}\right) A\times B - AB\,\gamma_{sub_1} H_1$$

$$Q_{ult} = (.033\times 12\times 34 + .4\times .068\times 7\times 40)7\times 7 - 7\times 7\times .033\times 12 = 1{,}016.4 \text{ KIP.}$$

$\underline{\text{DRAG IN PILES}}$ (FROM FIG. 13-10):

$$nQ_D = n\,ab\,\gamma_{sub} L = 9\times 3\times 3\times .033\times 12 = 32.1 \text{ KIP.}$$

$$\underline{\text{SAFETY FACTOR}} = \frac{nQ_G}{Q + nQ_D} = \frac{1{,}016.4}{440 + 32.1} = 2.2$$

$\underline{\text{SOIL PRESSURE ON TOP OF LAYER ③}}$ $\quad N_c = 5.53$

$$P_3 = \frac{440 + 32.1}{(2\times .5\times 9 + 7)^2} = 1.84$$

$\underline{\text{BEARING CAPACITY OF LAYER ③:}}$

$$1.3\,CN_c = 1.3\times .6\times 5.53 = 4.32 > 1.84 \text{ KSF.}$$

FIGURE 13-9
Example of Computation of Safety Factor of Pile Groups

(2) Cohesionless Soils. For a large group of piles in cohesionless soil, shear failure is highly unlikely at normal pile loads. For pile spacing less than three diameters, the group acts like a large pier, mobilizing side friction on the outer boundary of the group. For pile spacing exceeding eight diameters, individual resistance of each pile is mobilized. For a preliminary estimate of allowable pile loads in a bearing stratum, apply the rule for minimum spacing given in Table 13-3.

2. **DRAG ON PILES.** Where piles are driven through an upper compressible material into the bearing stratum, skin friction resists movement during load test or pile driving. In the prototype, consolidation of compressible strata may reverse skin friction throwing additional loads on the pile, which are transmitted downward to the bearing stratum. Evaluate potential drag in accordance with Figure 13-10.

a. **Effect of Sensitivity.** Drag is maximum in sensitive clays that consolidate on remolding. In this case the entire weight of compressible material between piles in a cluster may be applied as drag. For clays with a sensitivity of less than two or three, consolidation produced by remolding from pile driving is limited to a thin annular space surrounding pile, and weight transferred to the pile is small.

b. **Effect of Surcharge.** Placing fill at the ground surface increases drag by causing consolidation and soil movement downward relative to piles.

c. **Effect of Stratification.** Drag is accentuated if coarse grained soils are interlayered with compressible strata. Surface fill or remolding of the soft strata also causes the granular materials to move downward along the piles.

d. **Allowance in Design.** If drag will occur, evaluate point bearing resistance of the piles separately in load tests. Add drag force to design load to obtain total load in the bearing stratum. An overload by drag, up to 15 percent of the allowable working load, is permitted on the piles, when a safety factor of 2-1/2 to 3 is available for the working load.

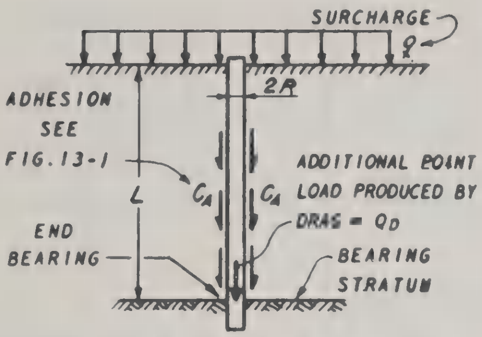
Section 5. PILE CLUSTER SETTLEMENT

1. **EVALUATION OF SETTLEMENTS.** For waterfront structures, a preliminary review of boring or site information may reveal an obvious need for pile foundations to form piers or resist scour. In other cases, settlement analysis of the greatest possible accuracy is required before eliminating shallow foundations from consideration.

a. **Point Bearing Piles.** Piles driven to a thick, compact, coarse grained stratum will not settle significantly, if an ordinary safety factor is applied to ultimate pile capacity.

b. **Piles Bearing Above Compressible Stratum.** Piles bearing in compact sand above a fine grained stratum may be safe against shear failure, but settlement must be analyzed as shown in Figure 13-11. Ordinarily, the increment of pressure applied by piles below their tips does not include drag forces or transient or temporary live load for settlement computation.

c. **Friction Piles in Loose Sand.** Utilize displacement piles that will compact surrounding soil. Driving should proceed from center of cluster to edge, otherwise the inner piles cannot be driven to an equal depth. Ordinarily, driving to a resistance for 20 tons for wood piles or 30 tons for concrete piles as determined by dynamic formula will ensure that settlements are within tolerable limits.



SINGLE PILE

SENSITIVE CLAY

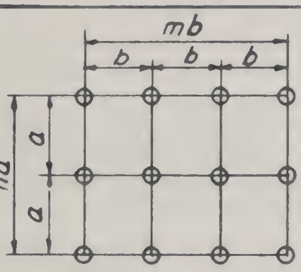
MAXIMUM DRAG FORCE:

WITHOUT SURCHARGE	WITH SURCHARGE
$Q_D = 2\pi R C_A L$	$Q_D = 2\pi R C_A L$

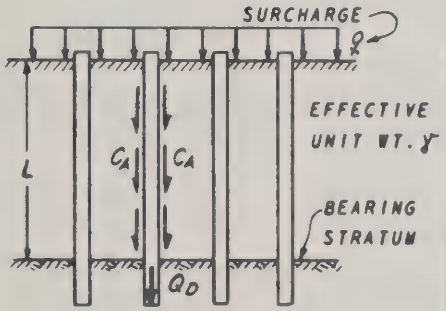
INSENSITIVE CLAY

MAXIMUM DRAG FORCE:

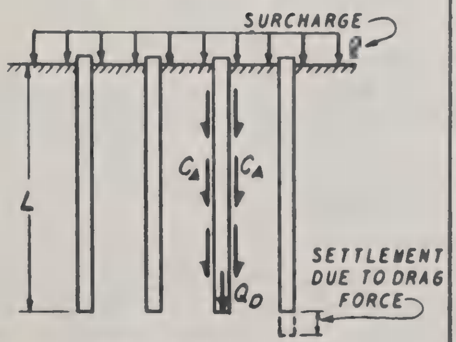
WITHOUT SURCHARGE	WITH SURCHARGE
$Q_D = 0$	$Q_D = 2\pi R C_A L$



PILE CLUSTER



BEARING PILES



FRICTION PILES

SENSITIVE CLAY

END BEARING PILES

MAXIMUM DRAG FORCE:

$$Q_D = ab\gamma L$$

BUT DRAG CANNOT EXCEED THE SMALLER OF THE FOLLOWING TWO FORCES:

$$Q_D = 2\pi R C_A L$$

OR

$$Q_D = \frac{2(na + mb)C_A L}{\underbrace{(n+1)(m+1)}_{\text{NUMBER OF PILES}}}$$

FRICTION PILES

WHEN PILE RELIES ON SIDE FRICTION FOR SUPPORT AND THE DIRECTION OF SIDE FRICTION IS CHANGED TO DRAG BY CONSOLIDATION, THEN DRAG FORCE THAT CAN DEVELOP IS LIMITED BY POINT BEARING RESISTANCE OF PILE BUT COULD EQUAL THE MAXIMUM DRAG FOR END BEARING PILES IF THIS VALUE IS LESS THAN POINT RESISTANCE.

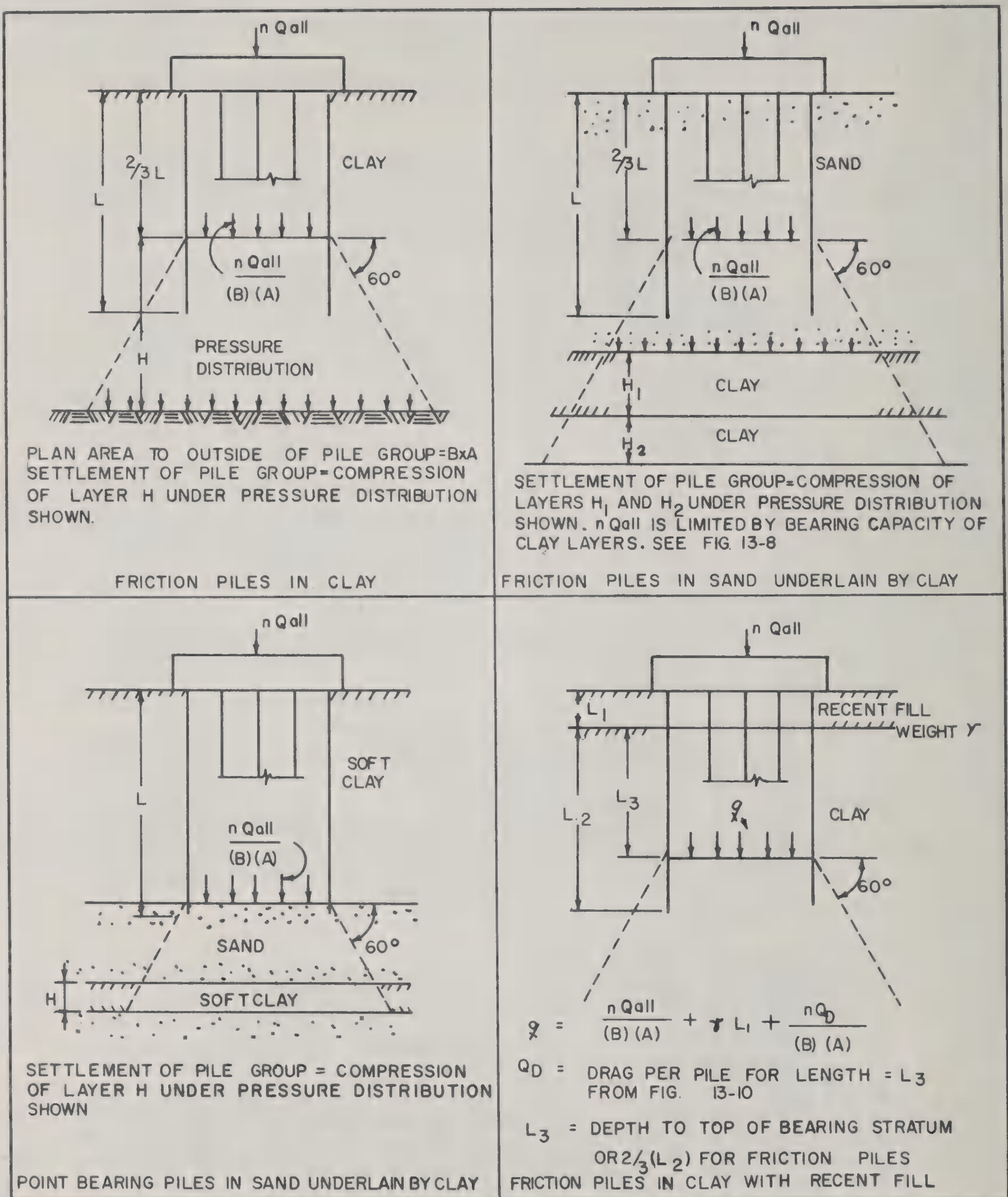
INSENSITIVE CLAY

END BEARING AND FRICTION PILES

WITHOUT SURCHARGE	WITH SURCHARGE
BECAUSE ONLY A LIMITED VOLUME OF REMOLDED SOIL IS CONSOLIDATING, DRAG IS SMALL COMPARED TO POINT BEARING CAPACITY OF PILE.	DRAG FORCE APPROACHES AS AN UPPER LIMIT THE VALUES FOR PILES IN SENSITIVE CLAY.

CONDITION	WITHOUT SURCHARGE	WITH SURCHARGE

FIGURE 13-10
Analysis of Drag on Piles in Clay



Notes: 1. Plan area to outside of pile group = B × 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 (Chapter 5) for load applied at level shown.
4. Compute settlements by methods of Chapter 6.

FIGURE 13-11
Settlement of Pile Groups

d. Friction Piles in Compressible Fine Grained Soils. Compute consolidation settlements for pressure distribution shown in Figure 13-11. The sole advantage of friction piles in minimizing settlements in clays is to lower the level at which loads are applied in the compressible stratum. This has the effect of stressing material that has a higher initial consolidation pressure than will the structure load applied at the surface. Because the reduction in settlement provided by friction piles can be small, the performance of alternative shallow foundations should be carefully evaluated. In sensitive clays, settlement of pile tips will be greatly increased by the effect of drag unless potential drag is included in the design load. In this case, the point bearing capacity must be capable of safely resisting both drag and ordinary working load. Where a recent fill surrounds the piles, drag will result in even, moderately compressible material. Settlement beneath pile tips is evaluated as shown in Figure 13-11 for this case.

Section 6. DISTRIBUTION OF LOADS ON PILE GROUPS

1. VERTICAL PILE GROUPS. Distribution of design load on piles in groups is analyzed by routine procedures given in the standard references as follows:

(1) For distribution of applied load eccentric about one or two axes, see Chellis, *Pile Foundations*.

(2) Overload up to 10 percent of allowable working load from eccentricity between applied load and center of gravity of pile group shall be permitted when a safety factor of 2-1/2 to 3 is available for the working load.

(3) Overload from wind 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.

2. GROUPS WITH VERTICAL AND BATTER PILES. Analyze distribution of pile loads according to Chellis, *Pile Foundations*. 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 Seelye, *Foundations, Design and Practice*.

(4) For analysis of batter pile anchorage for tower guys, see example, Figure 13-12.

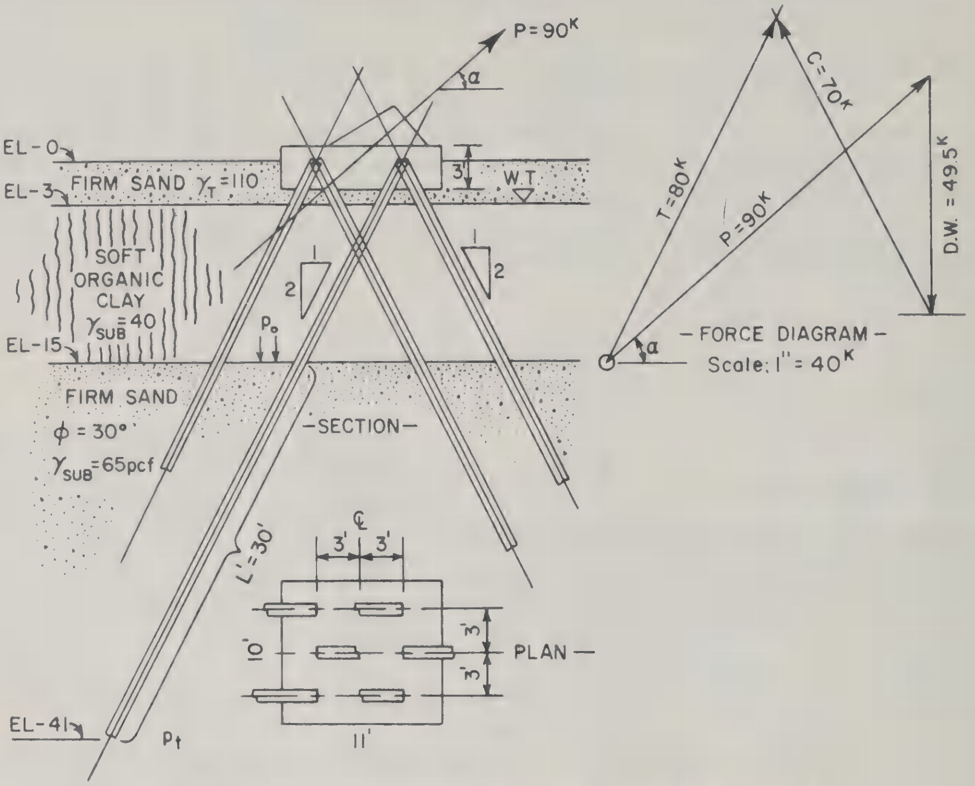
3. BUCKLING OF PILE GROUPS. For analysis of buckling of pile groups with an unsupported length above ground surface, see Abbett, *American Civil Engineering Practice*, Vol. I. If unsupported length is critical, reduce a long column by ties between piles or by batter piles.

Section 7. GENERAL CRITERIA

1. GEOMETRIC REQUIREMENTS. See Table 13-3 for criteria for pile spacing, length, number, arrangement, and alinement.

2. PRESERVATION AND DETERIORATION. For review of factors causing pile deterioration and methods of preservation, see Chellis, *Pile Foundations, Deterioration and Preservation of Piles*.

3. INSTALLATION PROBLEMS. Difficulties in installation of pile foundations are crucial in many cases, dictating pile type and arrangement. See Table 13-5 for common field problems and recommended treatment.



BATTER PILE GROUP ANCHORAGE FOR RESULTANT OF GUY FORCES (P):

SIX PILE GROUP, 4 TENSION, 2 COMPRESSION.

ASSUMING 10" SQUARE, 45' LONG, STRAIGHT CONCRETE PILES.

PILE CAP DEAD WT = 10' x 11' x 3' x 150 p.c.f. = 49.5^K.

DETERMINE PILE FORCES BY FORCE DIAGRAM, OR OTHER.

TRY L' = 30' IN FIRM EMBEDMENT.

ENTER GRAPH Ⓐ FIGURE 13-2a WITH AVG OVERBURDEN PRESSURE (P_{AVG}) AND VALUE OF ϕ :

$P_o = 3 \times 110 + 12 \times 40 = 810$ p.s.f. = press. @ TOP EMBEDMENT

$P_t = P_o + 26 \times 65 = 810 + 1690 = 2500$ p.s.f. = press. @ TIP OF PILE

$P_{AVG} = \dfrac{P_t}{2} = \dfrac{2500}{2} = 1250$ p.s.f. ; $\phi = 30° \therefore S = 650$ p.s.f.

$T_{ULT} = S \times$ SURFACE AREA = 650 x 30 x 3.3 SF/LF = 64,500 #

4 PILES x T_{ULT} = 4 x 64,500 = 258^K

$SF = \dfrac{258^K}{80} = 3.2 > 2.0$ REQ'D

CHECK COMPRESSION PILES: $Q_{ULT} = q \times A + S \times$ SURFACE AREA.

ENTER GRAPH Ⓑ FIGURE 13-2 WITH P_t & $\phi \therefore q = 47^K$

(47^K x .69 SF + 650 x 30 x 3.3 SF/LF) x 2 PILES = 194^K

$SF = \dfrac{194}{70} = 2.77 > 1.5$ REQ'D

FIGURE 13-12
Example of Tower Guy Anchorage by Batter Piles

TABLE 13-5
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 0.005 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.

a. Movement of Completed Piles. In driving through soft to medium stiff clays, movements may be large if the volume displaced by piles exceeds 2 to 3 percent of volume enclosed within the boundary of the pile foundation. Piles in stiff to hard cohesive soils or dense, coarse grained material generally are not driven deep enough to disturb adjacent piles. However, heave of ground surface immediately surrounding a pile may occur. Incomplete saturation of surrounding soil, particularly in soft organic material, will materially reduce heaving tendency. See Tables 13-2 and 13-5 for methods of decreasing pile displacement and controlling heave.

b. Damage to Bearing Material. Stiff and brittle clays, clay shales, or shales are vulnerable to fracturing or softening at the tips of completed piles during driving of adjacent piles. Specify test redriving of selected piles and redriving of open-end pipe piles in these materials.

c. Specification Requirements. Pile length and tip elevation shown in a construction contract should be based on exploration data and soil and pile tests. This evaluation is expressed by driving resistance requirements in the specifications, as follows:

(1) For piles bearing below a soft upper stratum, specify also a minimum depth of pile penetration.

(2) Where compressible materials are found below bearing stratum, specify a maximum permissible pile penetration.

(3) In coarse grained soils where scour or other considerations require deep penetration, jetting should not be permitted closer than 3 to 5 ft above pile tip elevation. Jetting near the pile tip should be carefully inspected and controlled by the field engineer.

CHAPTER 14. PRESSURES ON BURIED STRUCTURES

Section 1. INTRODUCTION

1. **SCOPE.** This chapter concerns analysis of pressures on buried or excavated underground structures. These structures include prefabricated or cast-in-place pipe placed in trenches, horizontal openings excavated for tunnels, and vertical shafts.

2. **RELATED CRITERIA.** For certain criteria not covered in this chapter but concerning the design of buried structures, see the following sources:

Subject	*Source*
Blast pressures on buried structures	NAVDOCKS P-81
Corrosion protection of pipes and conduits	NAVFAC DM-5
Electrical grounding requirements	NAVFAC DM-4
Pipe cover for airfield loads	NAVFAC DM-21
Pipe cover for highway loads	NAVFAC DM-5
Structural requirements of pipes and conduits	NAVFAC DM-5

Section 2. SHALLOW PIPES AND CONDUITS

1. **CONCRETE PIPE.** Precast or cast-in-place concrete pipes are relatively stiff and are loaded by vertical pressures that depend on pipe bedding, projection, and backfill conditions. Horizontal pressures vary from less than active for pipe bedding of poorest quality to greater than at-rest pressures for best pipe bedding. For methods of load analysis, see ASCE Manual of Engineering Practice No. 37, *Sanitary and Storm Sewers.*

2. **FLEXIBLE STEEL PIPE.** Corrugated or thinwall smooth steel pipes are sufficiently flexible to develop horizontal restraining pressures approximately equal to vertical pressures if backfill is well compacted. For load analysis of ordinary circular pipe use ASCE Manual of Engineering Practice No. 37. For design of large size flexible pipe of noncircular cross section see Armco Drainage and Metal Products, Inc., *Handbook of Drainage and Construction Products* (Bibliography).

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

a. Longitudinal Extension. The maximum horizontal strain of a conduit beneath an embankment or earth dam with narrow crest 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, where:

b = embankment base width,
h = embankment height above foundation at centerline,
d = thickness of compressible foundation below embankment centerline.

Approximate values of maximum horizontal strain at conduit midheight as percent of foundation vertical strain are as follows:

Values b/h	Values b/d 5	10	20	30
4	50 pct.	30 pct.	20 pct.	17 pct.
5	45	27	17	14
6	40	22	14	11
7	30	19	12	9

b. Joint Rotation. Besides the horizontal extension at midheight of the conduit, an additional joint opening occurs at the top or bottom of pipe from settlement beneath the pipe under embankment load. For concrete pipe in sections about 12 feet long, compute additional joint opening due to settlement by Equation 14-1.

$$\text{Opening} = \frac{\delta}{b}\, c \;\; r \qquad (14\text{-}1)$$

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 of Chapter 6. From this value, compute maximum joint opening at pipe midheight as maximum horizontal strain times length of pipe section. Add to this opening, the spread at top or bottom of pipe from joint rotation, computed from Equation 14-1. 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 concrete 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.

Section 3. TUNNELS

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

2. TUNNELS IN COHESIVE SOIL. Pressure analysis is complicated by the tendency of clay to undergo continuing displacements with constant or decreasing shear resistance.

a. Pressures During Construction. During construction, assume that full shear strength is mobilized on vertical planes extending upward from the tunnel and in the active wedge acting on the tunnel sides.

b. Pressures Following Construction. Following construction of shallow tunnels in plastic clays, vertical pressures approach the full weight of overburden. The ratio of horizontal to vertical pressure K_H ordinarily increases to values between one-half and two-thirds.

3. TUNNELS IN COHESIONLESS SOIL. Pressures depend on relative density, which determines friction resistance and magnitude of movement toward the tunnel. Generally, full shear resistance is mobilized both during construction and for the ultimate loads on the completed tunnel. See Terzaghi, *Theoretical Soil Mechanics* (Bibliography) for pressure analysis.

4. TUNNELS 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. See Bibliography for texts on rocks.

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 Obert, Duvall, and Merrill, *Design of Underground Openings in Competent Rock* (Bibliography). 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.

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 Terzaghi, *Rock Defects and Loads on Tunnel Supports* (Bibliography).

(1) Vertical Rock Load. The following table gives the height of rock above the tunnel roof which must be supported by roof lining, where b = tunnel maximum width, and h = tunnel maximum height.

Vertical Rock Load

Rock condition	*Equivalent vertical load (ft)*
Moderately blocky and seamy	0.25 to 0.35 (b + h)
Very blocky and seamy	0.35 to 1.0 (b + h)
Completely crushed, but chemically intact	1.10 (b + h)
Squeezing rock, tunnel at moderate depth	1.10 to 2.10 (b + h)
Squeezing rock, tunnel at great depth	2.10 to 4.50 (b + h)
Swelling rock	Up to 250

(2) Horizontal Pressures. Determine horizontal pressures acting on tunnel sides by applying the surcharge of this vertical rock load to an active failure wedge opposite the sides. Assume values of rock shear strength 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.

Section 4. VERTICAL SHAFTS

1. SHAFT IN SAND. In the excavation for a vertical cylindrical shaft in cohesionless 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.

a. Pressure Coefficients. See Figure 14-1 for active and passive pressure coefficients for a cylindrical shaft of unlimited depth in cohesionless soils.

b. 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 active pressures on a long wall with plane strain in the surrounding soil, see Chapter 10.

2. SHAFTS IN CLAY. For information concerning pressures on shafts in cohesive soils, see Terzaghi, *Theoretical Soil Mechanics* (Bibliography). Shear resistance, mobilized in the surrounding soil on the plane of the base of a finite depth shaft, can reduce active pressures to values about one-half of the theoretical pressures for an infinite depth shaft.

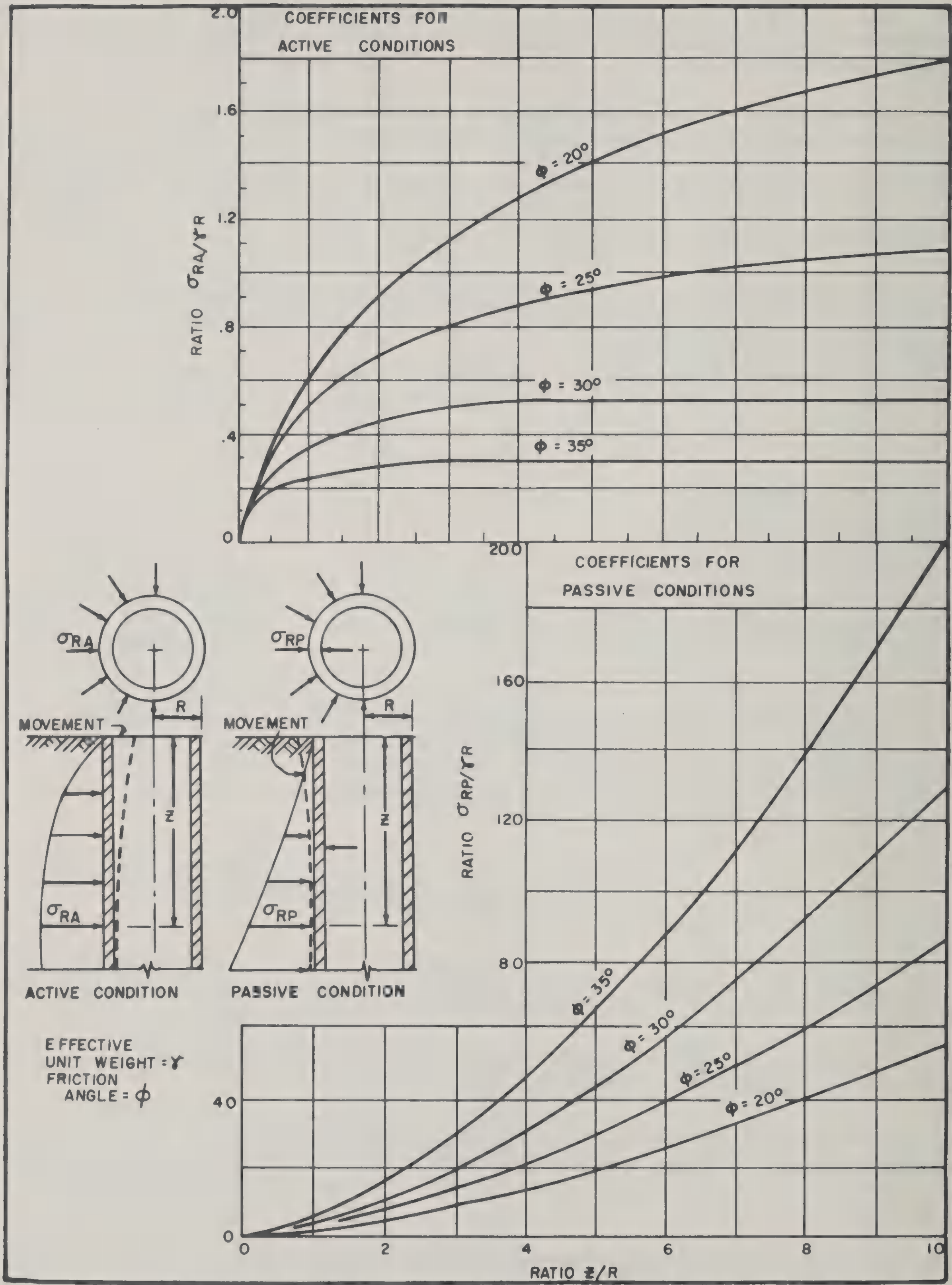


FIGURE 14-1
Coefficients for Active or Passive Pressures on Underground Cylindrical Shafts or Silos

CHAPTER 15. SOIL AND ROCK STABILIZATION

Section 1. INTRODUCTION

1. SCOPE. Materials, procedures, and applicability of methods for stabilizing soil or rock are described in this chapter. Methods include densification, drainage, changing soil properties in depth by grouting or injection, and surface stabilization by admixtures.

2. RELATED CRITERIA. For detailed criteria concerning stabilization for specific purposes, see the following sources:

Subject	*Source*
Densification by surface compaction	Chapter 9
Methods of decreasing or accelerating settlements	Chapter 6
Reservoir impermeabilization	Chapter 8
Slope stabilization	Chapter 7
Stabilization by drainage	Chapter 8
Stabilization for roads	NAVFAC DM-5

3. APPLICATIONS. Densification and drainage are utilized in a variety of problems. Injection at depth is undertaken to increase strength and rigidity of foundation materials or to decrease their permeability. Improvement of surface soils by admixtures is performed in highway and airfield construction, and for impermeabilization or erosion control.

Section 2. DENSIFICATION AND DRAINAGE

1. PROCEDURES. See summary in Table 15-1. Before adopting elaborate or unusual methods of stabilization, consider using ordinary densification or drainage techniques. Conditions requiring treatment by exploration and tests of samples recovered are defined as follows:

(1) Where low density of foundation is suspected, measure standard penetration resistance, unit weight, and moisture content in situ.

(2) Where drawdown will be required, determine the extent of pervious strata and piezometric levels within them.

(3) Estimate requirements for a drainage system from sample gradation data, laboratory or field permeability tests, and flow net analyses.

2. DENSIFICATION. Methods include compaction of material in thin lifts and procedures for increasing density at depth without applying surface load.

a. Surface Compaction. This is the common procedure for stabilizing earth structures and for improving natural soils near the ground surface.

b. Compaction in Depth. Three methods, summarized in Table 15-1, are used in coarse grained soils where pore water can be expelled rapidly without maintaining hydrostatic excess pressures. These methods are applicable when relative density is less than 70 to 75 percent. Compaction may not be needed at higher densities and may be difficult to achieve. Explosives and the driving of displacement piles have

TABLE 15-1

Stabilization by Densification or Drainage

Method	Procedure used	Applicability	Modification of soil properties
Surface compaction . .	Passage of rolling or vibrating compactors on fill built up in thin lifts.	Usual method for stabilization of earth structures. Control of compaction moisture is important in materials with more than 4 to 8 percent passing No. 200 sieve. Applications in foundation work depend on economy, time, available working area, or natural moisture conditions. Not suitable for highly organic soils.	Increase in density results in greater strength and rigidity. Stratification is destroyed and material is made more homogeneous and generally less pervious.
Compaction in depth by vibration.	Large vibrating spud is jetted into the ground. During withdrawal, water jets directed downward from the spud combined with vibrating action compact material below point of spud while sand is fed around spud from surface.	Greatest effect in relatively uniform coarse grained soils with less than about 10 percent passing the No. 200 sieve. In dirtier material, the excess water cannot be expelled to permit densification. Suitable above or below the ground water table.	Produces a cylindrical column of densified material 6 to 8 ft in diameter. Relative density of sands, which were originally 30 to 50 percent, may be increased to 60 or 70 percent.
Compaction in depth by blasting.	Explosion of numerous small dynamite charges at points in the interior of the soil mass. Generally 6 to 12 lb of 60 percent dynamite in each charge.	Most successful in clean, uniform, coarse grained material. Soil should be fully saturated, preferably inundated. Not suitable for dense well-graded soils.	Single shot affects an area 5 to 8 ft diameter. Relative density of 30 percent increased to 60 percent in typical case. Void ratios of 0.7 to 1.0 decreased to 0.6 and 0.8.
Compaction in depth by pile driving.	Short displacement piles or mandrel with temporary plug are driven. During withdrawal of mandrel, the hole is backfilled with soil.	Applicable in loose, coarse grained materials or in special cases in dry sandy silts and loess, which are unsaturated and have numerous air voids.	Radius of influence of compaction is about 5 times pile diameter. Relative density increased typically 30 percent in loose sand, 10 percent in medium compact.
Drainage accompanied by pumping.	Ground water level is drawn down by flow to deep wells, well points, or sumps while water is pumped from them.	Generally effective in materials with no more than about 25 percent smaller than 0.05 mm. In laminated or varved fine sandy silts or varved sand-silt-clay mixture, drawdown may be effective with as much as 50 percent of the average material smaller than 0.05 mm. Selection of method depends on arrangement of permeable strata, total depth of drawdown required, and character of excavation to be protected.	Increases rigidity and strength of material by increasing effective stresses acting in the soil. Prevents erosion and piping from breakout of seepage in the excavation. If sufficient time is available, drawdown will consolidate compressible silt-clay strata.

TABLE 15-1 (Continued)
Stabilization by Densification or Drainage

Method	Procedure Used	Applicability	Modification of soil properties
Drainage by natural gradient without pumping.	Drainage tunnels, trenches, pervious fills, relief wells, or augered horizontal drains are constructed to provide seepage interceptor at atmospheric pressure. Collectors are included to convey seepage from the drainage system.	Depending on gradients, arrangement of the system, and soil stratification, these methods are applicable to a wide variety of materials, including fine grained soils in which seepage is delivered by pervious lenses, seams, fractures, or cracks.	Ground water pressures reduced and effective stress increased with resultant increase in strength and rigidity. Fissures and cracks in stiff cohesive soils or soft rock are drained.
Reduction of hydrostatic excess pore pressures by drainage.	Vertical sand drains or sanded wellpoint holes under superposed load, or sanded wellpoint holes with vacuum seal are installed in compressible stratum. Pore pressures exceeding boundary pressures in the drain holes cause drainage of pore water.	Applicable to soft and compressible, unstable fine grained soils with high void ratio, including organic materials.	Accelerates drainage of pore water pressure by providing closely spaced drains at atmospheric or less than atmospheric pressure. This speeds consolidation and increase in shear strength.
Drainage by electro-osmosis.	Sets of anodes and wellpoint cathodes are driven into soil and an electrical potential is set up between them. Under the influence of electro-osmotic forces, pore water moves towards the well point cathodes. Usually arranged with anode nearest to excavation slope so as to reverse the direction of seepage forces.	Applicable to fine grained materials having 25 percent smaller than 0.05 mm but no more than 25 percent smaller than 0.002 mm, materials that cannot be stabilized by gravity drainage. Character of the clay minerals and dissolved salts affect economy and practicability of the method. Highly plastic clays require approximately ten times the energy input of nonplastic silts.	In compressible soils, positive pore pressures are changed to tension, resulting in consolidation and gain in strength. In relatively incompressible silts, stabilizing effects are comparable to ordinary drawdown.
Desiccation by air circulation.	May include forced circulation of hot air in tunnels or natural ventilation through drainage material with large void spaces.	Specialized procedure that may be applicable in some cases in treating relatively thin strata of unstable cohesive soils.	Capillary stresses are induced in clay by evaporation resulting in decrease in moisture content and increase in shear strength.
Desiccation by transpiration.	Surface of slope that undergoes shallow sloughing is planted with suitable grasses, shrubs, or other vegetation.	Applicable to all soil types for erosion control and particularly in fine grained materials with large variations in seasonal moisture content.	Besides the usual effects in controlling erosion, capillary stresses are induced by transpiration, resulting in a decrease in moisture content and producing a relatively thin skin of stiff material.

been used successfully to compact unsaturated loessial silts and fine sands of high void ratio containing vertical cracks. Disadvantages of the methods include relatively high cost, difficulty of obtaining uniform compaction, and inability to determine accurately the results achieved.

3. DRAINAGE. Methods include drawdown to drainage structures, reduction of hydrostatic excess pore pressures built up under load, and drainage by electro-osmotic forces or by desiccation.

a. Gravity Drainage. Materials stabilized range down to silt sizes, but also include stratified sand-silt-clay, or clay and rock with water-bearing fractures, fissures, or lenses.

b. Reduction of Hydrostatic Excess Pressures. The load of fill or of atmospheric pressure is applied at the ground surface and drainage of pore water is accelerated by vertical sand drains or sanded well-points. These methods are used in compressible, fine grained, and unstable soils including organic materials.

c. Drainage by Electro-osmosis. Soils treated include silt and clay too fine to be drained by gravity with a coefficient of permeability in the range of 2×10^{-4} to 0.02×10^{-4} fpm. Electro-osmosis develops tension in pore water, causing consolidation and gain in strength of compressible soils. Careful study of soil characteristics is required before application. Obtain specialized assistance to evaluate suitability of electro-osmosis.

d. Desiccation. Whether produced by air circulation or by transpiration from vegetation, effects of desiccation are relatively superficial and major stabilization is achieved in unusual cases only.

4. SPECIAL METHODS.

a. Inundation of Foundation. This method is used primarily in two situations, as follows:

(1) In conjunction with pumping from wellpoints to densify loose, coarse grained fills. Seepage directed downward towards wellpoints applies a consolidating force.

(2) For prewetting loessial silts and fine sands of low density and low natural moisture content. Inundation may result in a compression of about 4 to 8 percent of the original thickness of loose silts. Inundation of clayey loess may not be effective unless a surcharge is applied in conjunction with the wetting.

b. Balancing Pressure of Compressed Air. Compressed air is applied to stabilize excavation for tunnels and deep vertical shafts in the following special cases:

(1) The method is effective over a wide range of soil types but frequently applied to silts and clays near the liquid limit, or to fine sandy silts that are difficult to drain by gravity.

(2) In coarse sand and gravel, clay blanketing of open faces may be necessary to avoid air loss or blowouts.

(3) Generally compressed air is not applied for hydrostatic heads exceeding 50 psi.

Section 3. STABILIZATION IN DEPTH BY GROUTING OR HARDENING

1. APPLICATIONS. Grouting is undertaken to increase strength, or to decrease compressibility or permeability, and to seal large voids. The techniques are useful for a variety of structures either as a construction expedient or to alter permanently the character of the foundation. Grouting is one of the alternatives that should be compared critically for cost and applicability. Injection techniques are relatively complex and costly, and are justified only when simpler procedures must be rejected.

a. Selection of Method. Choice of method depends on penetration achieved in the foundation material, durability of grout in situ, improvement of foundation properties achieved, and comparative costs. Clay slurry and various artificial grouting materials of low viscosity penetrate fine grained soils. Their use may be limited by requirements for strength and durability.

b. Investigations Required. Because of the difficulty of formulating simple rules for selection of grouting methods, evaluation prior to construction is of vital importance. Make the following studies:

(1) Determine overall stratification from test borings, and identify loose or pervious strata and layers containing cracks or cavities.

(2) For seepage problems, determine gradation of representative samples and permeability by field tests.

(3) For major applications, perform test grouting at selected locations using alternative materials and techniques.

2. PROCEDURES. See Tables 15-2, 15-3, and 15-4. These methods include injection under pressure of a cementing material to intrude voids and harden in place, injection of soil particles into pervious materials, injection of a cementing mixture in preplaced aggregate, and mechanical mixing of soil and cementing agent in place.

a. General Grouting Types. There are three general types of grouting.

(1) Area Grouting. Low-pressure blanket or area grouting performed to seal and consolidate the foundation near the surface, or to fill large void spaces.

(2) High-pressure Grouting. Intermediate- or high-pressure grouting done at depth to seal fissures or small void spaces under high overburden.

(3) Contact Grouting. Contact grouting by injection of a slurry at the outer surface of a tunnel or structure to seal possible passages for water flow.

b. Pressures. Grouting pressures at the point of injection in soil generally are limited to the total weight of overburden. Overburden pressures frequently are exceeded in rock grouting. Avoid pressures that will produce heaving and wedging of the foundation, opening additional fissures.

(1) Rock Grouting Pressures. Rock grouting pressures at the point of injection range from 3/4 to 3 psi per foot of overburden above the level injected. Coefficients vary as follows.

Type of Rock	*Pressure (psi/ft)*
Highly fractured and broken rock	3/4
Sound stratified rock	2
Sound, massive igneous rock	3

(2) Pressure Reduction. During high-pressure grouting, reduce pressures rapidly when grout intake suddenly increases.

c. Field Controls. Carefully supervise grouting operations to make any necessary changes in grout mixes and procedures as work proceeds. Maintain detailed record of grout pressures and grout take in various sections of each hole.

(1) Impermeabilization Estimate. Specify pressure testing in selected grout holes or adjacent cored holes to estimate the impermeabilization achieved.

(2) Piezometric Levels. For a cutoff grout curtain, place observation wells or piezometers upstream and downstream of the curtain to observe buildup of piezometric levels across the grouted zone.

(3) Examining and Separating. Evaluate potential bleeding of mix by observation segregation of grout samples that have remained in airtight containers for 12 hours.

(4) Evaluating Strength. For major installations, obtain core boring samples to evaluate strength or grout penetration.

TABLE 15-2
Cement, Bituminous, and Chemical Grouts

Process	Materials, mixtures, and admixtures	Procedure	Applicability
Portland cement grouting.	Used with water-cement ratios generally between 1:1 and 4:1. Admixtures include bentonite, silica gels, pozzolans to reduce bleeding and segregation; lignosulphonates to increase pumpability; set accelerators such as calcium chloride.	Pumped under low pressure for blanket grouting or under high pressure for deep cutoff grouting. Water-cement ratio and type and amount of admixtures are varied in the field to alter consistency of mix and penetration. For increased penetration scalped Type III cement finer than about 0.03 mm is used.	The most common grouting material for decreasing permeability and increasing strength. Utilized in coarse grain soils with D_{10} size as small as about 1 mm. Penetrates loose sands with D_{10} of 0.8 mm or dense sands with 1.2 mm D_{10} size. Fissures can be grouted in the range of about 0.06 to 0.01 mm depending on pressures, water-cement ratio, and cement types. Not appropriate for grouting large voids with vigorous ground water movement.
Bituminous emulsions.	Bitumen particles with diameter between 0.001 and 0.005 mm are dispersed in water. Before injection, a substance such as an ester of formic acid is added which hydrolyzes to act as a coagulator.	Injected by grouting system. Speed of coagulation may be greatly influenced by chemical composition of soil or ground water so that careful control is necessary to obtain desired penetration.	Utilized to decrease permeability in sands with D_{10} size as small as about 0.1 mm. Increase in strength is relatively insignificant.
Single-stage sodium silicate grouting.	Sodium silicate with a setting agent such as sodium aluminate in water solution. May be combined in cement or soil-cement grout mixes.	Sodium silicate and setting agent are premixed in proportions to obtain setting time in a range from a few minutes to several hours. Mixture is injected in driven pipes or pipes in boreholes.	Used to decrease permeability in sands with D_{10} size as small as about 0.08 mm. Compressive strength of grouted sand equals about 100 psi.
Two-stage sodium silicate grouting.	Two materials consist of sodium silicate and calcium chloride.	Two fluids are injected successively. Reaction between them is almost instantaneous and calcium silicate is precipitated in soil voids. Rapidity of the chemical action requires care in the injection to avoid premature contact of the chemicals.	Penetrates sand with D_{10} size as small as 0.08 mm. Permeability is reduced. Compressive strength of grouted sand ranges from about 500 to 1,000 psi.
Acrylamide methylene bis acrylamide.	Generally used as 5 to 10 percent solution in water with catalyst controlling gel time such as ammonium persulfate and sodium thiosulfate.	Pumped through perforated injection point. Concentration of catalyst is varied for setting time from less than 1 minute to 1-1/2 hours. Performance and properties not completely evaluated.	Penetrates silt and sand with D_{10} size as small as 0.013 mm. Possibly applicable to grouting in moving ground water because gelling time can be made very short. Compressive strength of grouted sands ranges from 50 to 100 psi.
Chrome lignin.	Various combinations of a lignosulfonate and a hexavalent chromium salt, usually in combination with an acid salt and other reagents. Available in preblended and proportioned formulations for ease of application.	Pumped through injection points either as a single solution or as two solutions blended in a mixing manifold at one point of injection. Setting time from one minute to several hours varied by adjusting water content. Strength inversely proportional to water content.	Used to decrease permeability in sands with D_{10} size as small as 0.08 mm. Compressive strength of grouted sand ranges from 20 to 200 psi.

TABLE 15-3
Grouts of Soil and Soil-Cement Mixtures

Process	Materials, mixtures, and admixtures	Procedure	Applicability
Sand-cement grouting.	Typical sand-cement ratio of loose volume varies from about 2:1 to 10:1. Addition of bentonite or fly ash reduces segregation and increases pumpability. Water-cement ratios from about 2:1 to 5:1 by volume.	Pumped in conventional grout system with slush pumps of long stroke having large valve openings. Generally mixing facilities are larger than for neat cement grouts. Volume of water required for pumpability varies with the sand-cement ratio roughly in the range of equal volumes of water and sand to a volume of water equal to 1/3 volume of sand.	Used for grouting large foundation voids, for mud jacking, or for contact grouting on the periphery of a structure or tunnel, or used without pressure or at low pressure for slush grouting to fill surface irregularities in rock foundation on which embankment is to be placed. Grout will penetrate gravels depending on gradation of sand in mix. D_{10} size of gravels generally about 3/4 in. For usual mixes, strengths vary with water content from 100 to 700 psi.
Clay-cement grouting.	Typical clay-cement ratio of loose volumes varies from about 3:1 to 8:1. Water-clay ratios from about 3/4:1 to 2:1 by volume. Water-cement ratios from about 3:1 to 10:1 by volume.	Setting time is delayed and grouting can be continuous or intermittent without danger of losing the hole. Before mixing, clay should be screened to remove erratic larger particles.	Penetrates sand of approximately the same grain size as does neat cement grout. Use for grouting comparatively large voids where clay admixture is included for economy. Strength of mix depends on water-cement ratio and averages about 100 psi for typical mix.
Clay bentonite grouting.	Dispersed clay slurry with a flocculating agent such as aluminum sulphate to cause the suspension to coagulate after injection. Water-clay ratio determined by pumpability.	Slurry is produced by mixing and pumped through pressure piping. Experimentation is necessary to produce best mix that is pumpable and will flocculate. In a special case silt slurry was injected to consolidate loess with high void ratio and vertical cracks.	Used in materials down to fine sand size with a D_{10} between about 0.2 and 1.0 mm, depending on grain size of clay. Permeability is reduced, strength of the soil is affected only slightly. Process is relatively cheap and clay grouts will penetrate finer sand than cement grouts, but the clay may be removed by vigorous ground water flow.

TABLE 15-4
Special Injection Procedures

Process	Materials, mixtures, and admixtures	Procedure	Applicability
Sand-cement grouting of preplaced aggregate.	Mixture of cement, concrete, or plaster sand, with admixtures of expansion inducing fluidifier and pozzolànic grade fly ash used to improve pumpability. Sand/cement plus pozzolan ratio from 1:1 to 4:1. Coarse aggregate is generally plus 1/2 in. to maximum convenient size.	Mortar mix is pumped through pressure piping into preplaced coarse aggregate. Grout is always introduced at bottom of aggregate mass. For structural work, admix causing expansion of wet grout is essential to assure bond of mortar to aggregate and embedded items. For lean concrete, high-speed mixer (1,500 to 2,000 rpm) may be used to permit increase of sand-cement ratio up to 4:1 with water-cement ratio of less than 1.0.	Utilized where conventionally mixed and placed concrete is difficult, such as in heavily reinforced members; for high density concrete in complex configurations; and for very deep subaqueous concrete. Used in structural repair of concrete and masonry structures.
Mixed in-place piles of cement and soil.	Cement grout is used with typical water-cement ratios of 0.4 to 0.6 plus admixture to increase pumpability.	Hollow shaft auger or mixing head is rotated into soil while cement grout is pumped through shaft and outlet ports into soil. Injection and mixing occurs on both downward travel and withdrawal. Reinforcement may be inserted into pile before set.	Compressive strengths of pile material vary from about 100 psi for soft, fine grained soil to 2,000 to 3,000 psi for clean, coarse material. Used for repair work or temporary construction on soft soils; for permanent piles of low capacity supporting warehouse floors; for cutoff wall surrounding excavation using overlapping piles. Piles of medium load capacity are formed in loose, coarse soil.
Asphalt grouting.	Asphalt with a melting point between 165° and 175° F. Other thermoplastic materials that solidify on cooling have been utilized such as melted gypsum, pitch, sulfur, and resin.	Asphalt heated to 470° to 500° F before entering pump. Pumped through pressure piping that may have to be electrically heated in cold weather.	Utilized to fill large foundation cavities where ground water velocity is sufficient to disperse other grouts. Asphalt contracts on cooling in place so that injection must be repeated to insure water-tight barrier.
Electro-chemical hardening.	Use of aluminum electrodes in the electro-osmosis process.	Direct current is passed between aluminum electrodes placed in the ground. Cations of alkali metals in soil are replaced by those of hydrogen and aluminum, reducing swelling properties, lowering liquid limit. Sodium cations are carried toward the cathode where they are deposited as aluminates.	Applicable in fine grained soils with clays having capacity for base exchange. Addition of calcium chloride through porous hollow electrodes appears to accelerate base exchange and hardening of clays. Procedures are still largely experimental and require specialized assistance.

3. SPECIAL METHODS. In addition to procedures in Tables 15-2, 15-3, and 15-4, numerous combinations of the ordinary materials or of special grouts have been utilized.

a. Grouting Combinations. Chemical grouts may be combined with soil or cement slurry to economize or to penetrate variable void spaces of stratified or heterogeneous materials. Outer limits of holes may be grouted with cement or sand-cement mixes, and interior rows injected with chemicals or bituminous emulsion for cutoff in coarse grain foundations.

b. Freezing. Stabilization by freezing has been performed as a construction expedient in excavation and shaft sinking where compressed air is not practical, or where soils are too fine grained to be drained by gravity or so pervious that the flow cannot be controlled. Methods include circulation of chilled calcium chloride brine in boreholes, expansion of carbon dioxide into a circuit of freezing pipes, or direct application of solid carbon dioxide and alcohol. Freezing is generally very costly and is utilized only under the most difficult conditions after a thorough review of alternative procedures.

Section 4. SURFACE STABILIZATION BY ADMIXTURES

1. METHODS AND MATERIALS. Surface stabilization by admixtures is used primarily for improvement of pavement base courses or for low-cost wearing surfaces (Table 15-5). Surface stabilization should be considered for low-cost warehouse floors and outside storage areas. For details of selection and utilization of admixtures and sources of data, see Woods, *Highway Engineering Handbook.*

a. Mechanical Stabilization. The process of mechanical stabilization consists of blending different soil or aggregate fractions to increase density and rigidity of the composite material and is applied principally to improve quality of base course, or for wearing surface of secondary roads.

(1) Base Course Compaction. Consider increased compaction of base course in lieu of the addition of a fine fraction that makes the composite material sensitive to frost action.

(2) Binder and Aggregate Blending. Consider blending binder and aggregate to decrease permeability of coarse grain soils for reservoir linings.

b. Methods in Development. When selecting surface stabilization methods using various chemicals or industrial process byproducts that bond soil particles, the designer should review methods under development. The use of inexpensive bonding materials for temporary roadway or beach stabilization is expedient in many cases.

2. IMPERMEABLE RESERVOIR LININGS. Various admixtures may be used to provide relatively impervious and flexible soil linings for reservoirs where tractive forces or wave action are insignificant. For specific procedures, see Table 8-2.

TABLE 15-5
Surface Stabilization With Admixtures

Admixture	Quantity, percent by weight of stabilized soil	Process	Applicability	Effect on soil properties
Portland cement.	Varies from about 2½ to 4 percent for cement treatment to 6 to 12 percent for soil cement.	Cohesive soil is pulverized so that at least 80 percent will pass No. 4 sieve, mixed with cement, moistened to between optimum and 2 percent wet, compacted to at least 95 percent maximum density, and cured for 7 or 8 days while moistened with light sprinkling or protected by surface cover.	Forms stabilized subgrade or base course. Wearing surface should be added to provide abrasion resistance. Not applicable to plastic clays.	Unconfined compressive strength increased up to about 1,000 psi. Decreases soil plasticity. Increases durability in freezing and thawing but remains vulnerable to frost effects.
Bitumen.	3 to 5 percent bitumen in the form of cutback, asphalt emulsion, or liquid tars for sandy soils. 6 to 8 percent asphalt emulsions and light tars for fine grain materials. For coarse grain soils, antistrip compounds are added to promote particle coating by bitumen.	Soil is pulverized, mixed with bitumen, solvent is aerated and mixture compacted. Before mixing, coarse grained soils should have moisture content as low as 2 to 4 percent. Water content of fine grained soils should be several percent below optimum.	Forms wearing surface for construction stage, for emergency conditions or for low-cost roads. Used to form working base in cohesionless sand subgrades, or for improving quality of base course. Not applicable to plastic clays.	Provides binder to improve strength and to waterproof stabilized mixture.
Calcium chloride .	1/2 to 1-1/2 percent	Normally applied at rate of about 0.5 lb/sq yd area. Dry chemical is blended with soil-aggregate mixture, water added and mixture compacted at optimum moisture by conventional compaction procedures.	Used as dust palliative. Stabilized mix of gravel-soil binder calcium chloride forms wearing surface in some secondary roads.	Retards rate of moisture evaporation from the stabilized mixture, tends to reduce soil plasticity. Greatest effect in sodium clays with capacity for base exchange. Lowers freezing point of soil water, decreasing loss in strength from freezing and thawing.
Lime	4 to 8 percent. Fly ash, between 10 and 20 percent, may be added to increase pozzolanic action.	Lime is spread dry, mixed with soil by pulvi-mixers or discs, mixture compacted at optimum moisture to ordinary compaction densities.	Used for base course and subbase stabilization. Generally restricted to warm or moderate climates because the mixture is susceptible to breakup under freezing and thawing.	Decreases plasticity of soil, producing a grainy structure. Greatest effect in sodium clays with capacity for base exchange. Increases compressive strength up to a maximum of about 500 psi.

TABLE 15-5 (Continued)
Surface Stabilization with Admixtures

Admixture	Quantity, percent by weight of stabilized soil	Process	Applicability	Effect on soil properties
Chrome lignin and sulfite lignin.	3 to 10 percent of sulfite waste liquor used with coarse grain soils. Sodium bichromate or potassium bichromate added to stabilize silt and clay or to prevent leaching of lignin from sands and gravels.	Soil and admixture may be mixed in place, or when bichromate is used, synthetic aggregate may be produced by briqueting the soil with the admixture and crushing the briquets.	Sulfite waste alone has been used to stabilize gravelly sand with up to 10 percent fines. Addition of bichromates permits stabilization of fine grain soils.	Mixture with bichromates is water repellent and relatively impervious. Bonds formed between additive and soil particles provide some strength and rigidity. Sulfite waste alone is water soluble and reapplications are necessary.
Methods under development:				
Rosin and various salts of abietic acid.	1 to 3 lb/sq yd for a 6-in. depth.	Soil is pulverized for the depth of stabilization, mixed with rosin and water, and compacted.	Applicable to soils with PI less than about 15. Soils to be treated must possess some stability when dry.	Rosin coats soil particles and acts as water repellent, but furnishes no cohesive bond.
Aniline furfural .	2 to 3 lb/sq ft for 6-in. depth.	Material mixed in pug mill by spraying chemicals. Mixture passes from pug mill into windrow and is compacted by a traveling vibrating compactor. Various pulverizing and mixing equipment may be utilized.	Stabilization of beach sands. Full traffic can be carried after 8 hours.	Provides a quick setting binder resistant to effect of water.
Calcium acrylate	1 to 2 lb/sq ft for 3-in. depth. Sodium thiosulfate and ammonium persulfate used as catalysts.	Chemicals premixed in aqueous solution and applied to soil.	Used for emergency wearing surface in coarse soils or fine grain material with moisture content up to about 30 percent.	Binder forms rigid surface when dry, rubbery when wet. Rigidity is recovered on drying. Surface is rubbery and elastic for 2 to 5 hours after treatment.

CHAPTER 16. SPECIAL PROBLEMS

Section 1. INTRODUCTION

1. **SCOPE.** This chapter is concerned with foundation design for seasonal frost areas, and foundation problems presented by vibrating loads and seismic forces.

2. **RELATED CRITERIA.** Additional criteria relating to frost or dynamic problems appear in the following sources:

Subject	*Source*
Blast loading on structures	NAVDOCKS P-81
Depth of frost	NAVFAC DM-2
Design problems in permafrost and arctic areas	NAVFAC DM-9
Fundamental aspects of frost action	NAVFAC DM-9
Pavement design for frost action	NAVFAC DM-21
Seasonal frost penetration beneath paved areas	NAVFAC DM-21
Seismic loading on structures	NAVFAC DM-2 & NAVDOCKS P-355

Section 2. FOUNDATIONS IN SEASONAL FROST AREAS

1. **MECHANICS OF FROST ACTION.** In seasonal frost areas, ground freezing occurs during winters followed by thaw without development of permafrost. The problems involved are vertical or horizontal expansion of soil during freezing and decrease of soil shear strength and rigidity during thawing. Significant heave and increase of soil moisture is caused by movement of supercooled pore water from small voids to ice crystals in larger voids where water solidifies, crystals grow, and ice lenses form.

a. Frost-susceptible Soils. Inorganic soils which are more than 3 percent by weight finer than 0.02 mm generally are frost susceptible and experience ice crystal growth, ice lensing, and heave in various degrees.

(1) *Capillary Soils.* Materials with small voids are more frost susceptible because of their greater capillary potential. The most pervious of these soils are the most susceptible because of their ability to transmit appreciable water through void spaces. Thus, silts, silty sands, and lean clays with PI less than 12 undergo the largest frost heave.

(2) *Clays.* Clays of medium to high plasticity are susceptible to the formation of ice lenses, but significant heave develops only at long, sustained freezing temperatures when there is enough time for water migration.

(3) *Coarse Grained Soils.* Dense, broadly graded, coarse grained soils with 1½ to 3 percent by weight finer than 0.02 mm are of borderline susceptibility and must be tested to evaluate their performance.

b. Nonfrost-susceptible Soils. Clean GW, GP, SW, and SP materials with 1 percent or less smaller than 0.02 mm and uniform sands with 10 percent or less by weight smaller than 0.02 mm are nonfrost susceptible. However, soils frequently are interbedded with other soils and usually cannot be considered separately.

c. Influence of Water Conditions. Heave is limited if freezing soil has no access to free water, other than that in its voids, immediately below the freezing plane.

(1) Water Sources. Sources of water for ice segregation are found in the following:

(a) Coarse grained soils, where capillary potential may draw water from ground water as deep as about 5 feet below the freezing plane.

(b) Silts, varved silts and clays, or silty, very fine sands, where capillary supply may be effective with ground water as deep as about 10 feet below the freezing plane.

(c) Saturated compressible clays, where sufficient moisture for ice segregation may be extracted from void spaces by compression of the underlying clay even if ground water is more than 10 feet deep.

(d) Water is provided by infiltration resulting from poor surface drainage, leakage of water or sewer lines, or perched water in pockets or lenses of sand supplied by surface sources.

(2) Frost Heave Reduction. Potential frost heave is reduced by surface or subsurface drainage which lowers the ground water, decreases the degree of saturation of surface soils, or prevents surface infiltration.

d. Depth of Frost Penetration. Penetration depends on weather, surface cover, soil properties, and moisture. For average annual frost penetration depths in the United States, see Abbett, *American Civil Engineering Practice*, Vol. 1, and NAVFAC DM-2, Table 1-6. To estimate penetration in developed areas, refer to building codes or local usage. Values given may suffice for average cover and soils, but are generally somewhat less than deepest penetration recorded in the last 20 to 30 years. Compute frost penetration for unusual conditions, or for remote areas where empirical data are lacking, by methods of Arctic and Subarctic Construction, U.S. Army Corps of Engineers, Technical Manual TM 5-852-6. Frost penetration for unheated structures surrounded by cleared, paved areas is similar to that for pavements, as determined by methods given in NAVFAC DM-21.

2. SPREAD FOUNDATIONS. Place spread foundations for permanent structures below frost penetration for coldest year in latest 30 years of record. For temporary structures, use coldest year in latest 10. For heated buildings, both slab-on-grade and basemented, reduce these requirements on the basis of local experience and codes. However, exterior footings for support of porches, roof extensions, or unheated connecting corridors are subject to full frost heave.

a. Footing Posts. A bond of frozen soil with concrete posts will develop tension in footing posts during heave. This adfreeze grip ranges from 50 to about 300 psi. To protect posts or columns, anchor footings by spreading base below frost penetration and include suitable reinforcement in post. To minimize adfreeze, batter the faces in contact with frost-susceptible soil as much as practicable, extending the batter well below frost penetration. Form small posts of pile columns surrounded by protective casing through frost zone. Fill space between column and casing with an oil-wax mixture.

b. Exterior Aprons. For exterior slabs whose heave may block doors, provide a pad of nonfrost-susceptible material to the depth of frost penetration if the pad can be drained. If this is impracticable, support structural slab on building foundation, spanning to grade beam in natural soil on opposite side, leaving space beneath slab to accommodate heave.

c. Unheated Buildings. For an unheated slab-on-grade building, there are several foundation alternatives.

(1) Fill Replacement. Replace frost-susceptible material to a depth of frost penetration (for the coldest of the latest 10 years) with compacted, clean, coarse grained fill. A buried layer of cellular glass insulation will reduce thickness of select material required.

(2) Piles. Drive piles to sufficient depth to resist frost heave, supporting the floor above ground. The depth of piles may be reduced by casings which eliminate adfreeze grip on piles.

(3) Columns. Use a steel or concrete column, tied to footings below frost penetration with floor supported above ground.

(4) *Foundation Walls.* Use conventional footing or wall foundations at the bottom, below frost penetration designed to resist adfreeze grip. Replace grade beams subject to uplift with full foundation walls.

d. Drainage and Utilities. To minimize heave, slope ground down away from building and convey drainage from roofs and concrete surfaces in gutters for a distance of at least 10 feet from the building. Place water and sewer lines 6 inches below frost penetration. Insure that utility lines will not be broken by heave at foundation walls.

e. Construction Precautions. Protect spread foundations during freezing periods if the frost-susceptible soil underlies the structure within the depth of possible frost penetration. Avoid supporting posts for concrete formwork on the surface of frost-susceptible material.

3. **WALLS AND RETAINING STRUCTURES.** Horizontal frost penetration in a frost-susceptible backfill in contact with an unheated wall will cause water migration, ice lensing, and lateral thrust. If the backfill forms an open system with access to unlimited water, this thrust can move or break the wall. No serious frost action will develop if backfill is completely drained and surface infiltration prevented. See Chapter 8 for wall drainage.

Section 3. VIBRATION PROBLEMS

1. **DESIGN CONSIDERATIONS.**[1] Generation or transmission of periodic vibrations presents three foundation problems:

a. Settlements. Vibration tends to densify loose nonplastic soils, causing settlement. The maximum effect occurs in clean, coarse grained materials. Loose or medium compact sands must be compacted to receive spread foundations for vibrating equipment; see methods of Chapter 15. Shock or vibrations near a foundation on loose, saturated nonplastic silt, or silty fine sands, may produce a quick condition and loss of bearing capacity. In these cases, bearing intensities should be less than ordinary values for static loads. For severe vibration conditions, reduce bearing pressures to one-half of allowable static values.

b. Resonance. Settlements from vibratory loads are accentuated if imposed vibrations are resonant with the natural frequency of the foundation soil system. Both amplitude of foundation motion and unbalanced exciting force are increased at resonance, and even compact cohesionless soils will be densified with accompanying settlement. Design foundations for vibrating machinery to avoid resonance by methods given in Paragraph 3. Avoidance of resonance is particularly important to prevent settlement in cohesionless materials, but should be considered for all types of foundation soils to avoid structural damage.

c. Vibration Transmission. Transmission of vibrations originating outside a structure or from machinery within the structure may be annoying to occupants and damaging to the structure, or may interfere with the operation of sensitive instruments. See the upper panel of Figure 16-1 for the combined effect of vibration amplitude and frequency. Tolerable vibration amplitude decreases as frequency increases. No value of allowable amplitude should be considered as a design requirement unless the frequency is also specified. For methods of reducing amplitude of vibrations transmitted into a structure or away from a vibrating source, see Paragraph 4.

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[1]The Bibliography lists references which may provide additional background and solutions for vibration problems. They should be studied for guidance, as supplementary to the material in this manual.

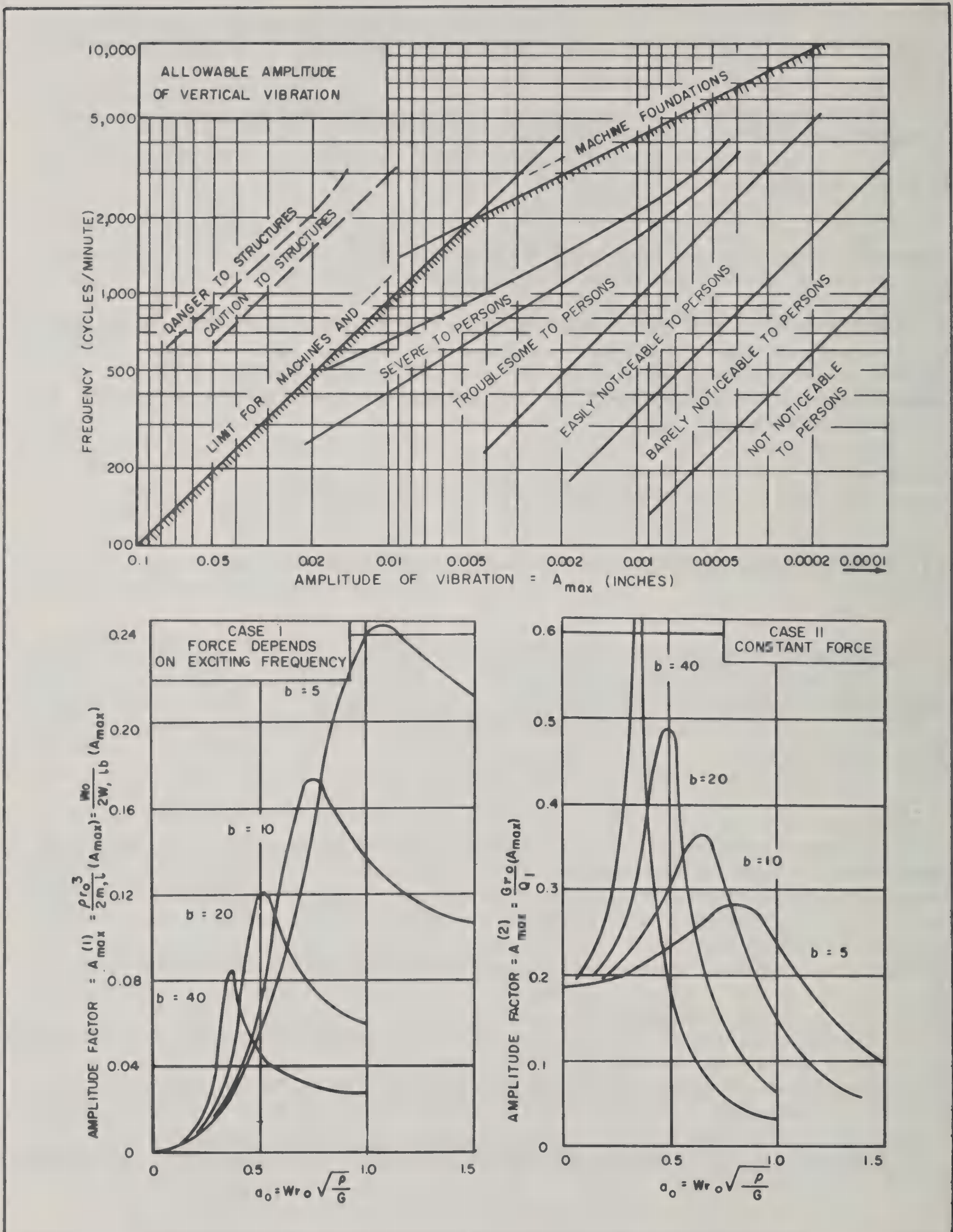


FIGURE 16-1
Relationship Between Amplitude and Frequency of Oscillating Load

2. DYNAMIC SOIL PROPERTIES. Natural frequencies vary from about 400 cpm for soft organic soils and loose fine sand, to 2,000 cpm for sedimentary rocks. Corresponding values of dynamic modulus of shear rigidity G range from about 1,500 to 3,500 psi with Poisson's ratio μ from 0.25 to 0.50. Machinery operating in this frequency range includes reciprocating engines, compressors, and large blowers.

a. Machinery Characteristics. The most troublesome resonance problems occur with equipment operating at speeds less than 1,000 rpm. The most frequent difficulties are with reciprocating engines working at about 400 rpm, founded on soft or loose soils.

b. Determination of Soil Properties. Determine dynamic soil properties for specific sites by seismic methods, or with a test oscillator operating at frequencies and amplitudes comparable to those anticipated for the prototype.

3. DESIGN TO AVOID RESONANCE. Figures 16-2 and 16-3 illustrate the method of selecting size and weight of machinery foundation block to avoid resonance with the vibrating equipment. In general, natural frequency of the foundation block soil system should be less than one-half or greater than two times the equipment operating frequency. Increase of foundation weight decreases natural frequency. Increase of foundation contact area (equivalent to a decrease in bearing pressure), or stiffening the subsoil by grouting or injection, increases the natural frequency. The allowable amplitude of motion at operating frequency is given for various requirements in the upper panel of Figure 16-1.

a. Characteristics of Vertical Oscillations. Ordinarily, vibrations are produced by vertically oscillating loads of two types; when the force produced depends on the angular velocity of movement of the unbalanced mass, and when force is independent of frequency of oscillator. The first condition, Case I, is represented by reciprocating engines. The second condition, Case II, is for impact vibrations such as produced by hammers.

b. Maximum Amplitude of Vibration. For given subsoil properties and vibrating weight, determine resonant frequency and maximum amplitude of oscillation from Figure 16-4. For complete relation of amplitude to frequency, see bottom panels of Figure 16-1. These diagrams apply to an oscillator with a rigid circular base resting on a semi-infinite elastic medium. Determine for design the unbalanced force and operating frequencies of the machine, the shear modulus G, Poisson's ratio μ, and the total unit weight of foundation soil. If the foundation block has a base shape that differs markedly from a circle, or if the foundation is embedded to an appreciable depth, consider subgrade as a weightless spring, and determine ratio of resonant frequencies on real to weightless medium from mass ratio; see top panel of Figure 16-5. Apply this ratio to the resonant frequency of Figure 16-4 to determine resonant frequency for weightless foundation medium.

c. High-Speed Machinery. For machinery with operating speeds exceed about 1,000 rpm, provide a foundation with natural frequency no higher than one-half of the operating value, as follows:

(1) Decrease foundation frequency by increasing the foundation block weight in accordance with formulas of Figure 16-2.

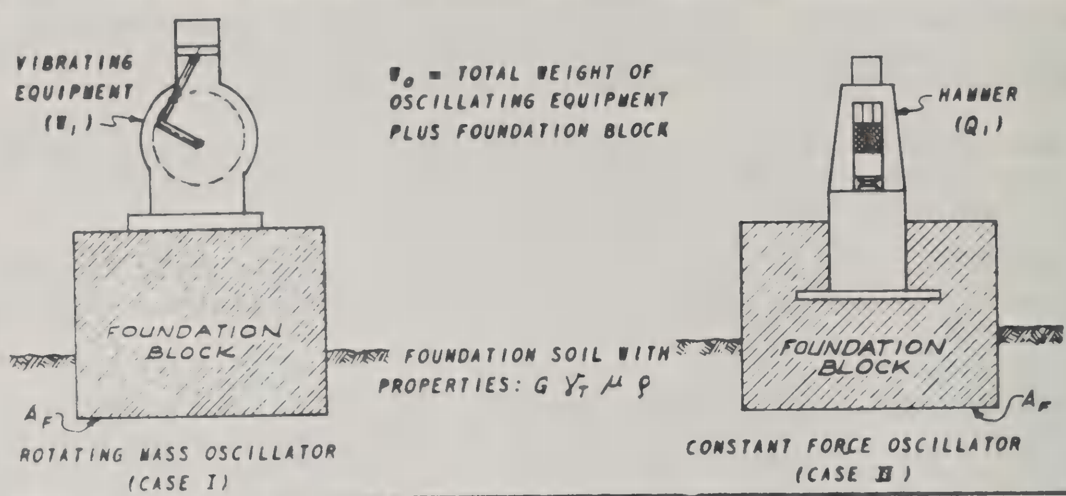
(2) During starting and stopping, the machine will operate briefly at resonant frequency of the foundation. Compute probable amplitude at both resonant and operating frequencies, and compare them with allowable values of top panel of Figure 16-1 to determine if the foundation arrangement must be altered.

d. Low-speed Machinery. For machinery operating at a speed less than about 300 rpm, provide a foundation with a natural frequency at least twice the operating speed, as follows:

(1) For spread foundations, increase its natural frequency by increasing base area or reducing total static weight.

(2) As an alternative, increase modulus of shear rigidity of the foundation soil by compaction, grouting, or injection; see Chapter 15 for methods.

(3) Consider the use of piles to provide the required foundation stiffness. See bottom panel of Figure 16-5 for natural frequency of various point bearing piles.



DEFINITIONS

A_{max} = MAXIMUM AMPLITUDE OF OSCILLATION.
$A^{(1)}_{max}$ = DIMENSIONLESS AMPLITUDE FACTOR CORRESPONDING TO AMPLITUDE PRODUCED BY A ROTATING MASS OSCILLATOR (CASE I).
$A^{(2)}_{max}$ = SAME AS ABOVE FOR A CONSTANT MASS OSCILLATOR (CASE II).
A_f = AREA OF FOUNDATION BASE.
a_o = DIMESIONLESS FREQUENCY FACTOR.
b = DIMENSIONLESS MASS RATIO.
f_0 = RESONANT FREQUENCY OF VIBRATING EQUIPMENT, FOUNDATION BLOCK, AND FOUNDATION SOIL AS A UNIT.
f_1 = OPERATING FREQUENCY OF THE VIBRATING EQUIPMENT.
f_W = SAME AS f_0 ON WEIGHTLESS MEDIUM.
F_f = VERTICAL FORCE AT f DUE TO W_1.
G = SHEAR MODULUS OF ELASTICITY
g = ACCELERATION OF GRAVITY.
l = RADIUS FROM CENTER OF GRAVITY OF ROTATING PART TO CENTER OF ROTATION.
m_0 = TOTAL STATIC MASS OF VIBRATING EQUIPMENT.
m_1 = UNBALANCED ROTATING MASS WITHIN VIBRATING EQUIPMENT.
r_0 = RADIUS OF CIRCULAR BASE OF FOUNDATION BLOCK.
W_0 = TOTAL STATIC WEIGHT OF VIBRATING EQUIPMENT PLUS FOUNDATION BLOCK.
W_1 = UNBALANCED FORCE OF ROTATING MASS OSCILLATOR
γ_T = UNIT WEIGHT OF FOUNDATION SOIL.
μ = POISSON'S RATIO OF FOUNDATION SOIL.
ρ = MASS DENSITY OF FOUNDATION SOIL.
ω = ANGULAR VELOCITY AT OPERATING FREQUENCY.
f = FREQUENCY OF VIBRATION IN GENERAL.
F_{f_0} = VERTICAL FORCE AT f_0 DUE TO W_1.
Q_1 = UNBALANCED FORCE OF CONSTANT FORCE OSCILLATOR.
E = SOIL COMPRESSION MODULUS, psi

FORMULAS

$$A_{MAX} = \frac{2 m_1 l}{\rho r_0^3} A^{(1)}_{MAX} \qquad A_{MAX} = \frac{W_1}{G r_0} A^{(2)}_{MAX}$$

$$a_0 = \omega r_0 \sqrt{\frac{\rho}{G}} = 2\pi f_0 r_0 \sqrt{\frac{\rho}{G}} \qquad b = \frac{W_0}{\gamma_T r_0^3}$$

$$f_0 = \frac{a_0}{2\pi r_0} \sqrt{\frac{G g}{\gamma_T}} \qquad f_W = \frac{1}{\pi} \sqrt{\frac{G r_0 g}{W_0 (1-u)}}$$

$$F_{f_0} = W_1 \left(\frac{f_0}{f_1}\right)^2 \qquad F_f = 2 m_1 l \omega^2$$

$$m_0 = \frac{W_0}{g} \; ; \quad m_1 = \frac{W_1}{g}$$

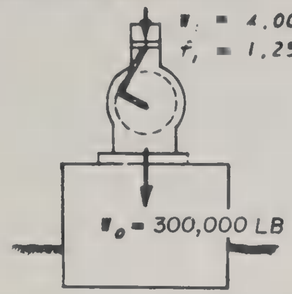
$$r_0 = \sqrt{\frac{A_F}{\pi}} \qquad \rho = \frac{\gamma_T}{g} \qquad \omega = 2\pi f_1$$

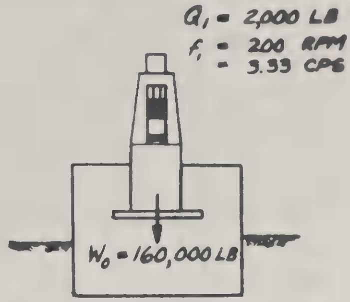
$$G = \frac{E}{2(1+\mu)}$$

DESIGN PROCEDURE

1. DETERMINE THE CHARACTERISTICS OF THE VIBRATION TO BE CONTROLLED, INCLUDING UNBALANCED WEIGHT OF VIBRATING EQUIPMENT.
2. DECIDE ON PERMISSIBLE AMPLITUDE OF VIBRATION. UPPER DIAGRAM OF FIG. 16-1 MAY BE USED AS A GUIDE.
3. DETERMINE PROPERTIES OF FOUNDATION SOIL: E γ_T μ.
4. DETERMINE WEIGHT OF MACHINE AND ASSUME SIZE AND WEIGHT OF FOUNDATION BLOCK.
5. EVALUATE FREQUENCY AND MAX. AMPLITUDE OF FOUNDATION BLOCK-SOIL SYSTEM, SEE FIG 16-3.

FIGURE 16-2
Basic Relationships for Foundation Vibrations





ROTATING MASS OSCILLATOR (CASE I)

GIVEN A HIGH SPEED GENERATOR WITH THE FOLLOWING CHARACTERISTICS:

$W_0 = 300{,}000 \qquad W_1 = 4000$ LB

$f_1 = 1{,}250$ RPM

FOUNDATION SOIL VALUES FROM TESTS:

$\gamma_T = 120$ PCF $\qquad E = 10{,}000$ PSI

$\mu = 0.35 \qquad G = 3{,}700$ PSI

DESIGN THE FOUNDATION SO THAT THE RESONANT FREQUENCY (f_0) OF VERTICAL OSCILLATION IS HALF THE OPERATING FREQUENCY (f_1) AND SO THE AMPLITUDE OF VIBRATION (A_{MAX}) IS LESS THAN 0.0004 IN.

ASSUME $A_f = 18 \times 14 = 252$ SQ FT

$r_0 = \sqrt{\dfrac{252}{\pi}} = 8.95$ FT $\qquad r_0^3 = 717\ FT^3$

1. RESONANT FREQUENCY:

$$b = \frac{W_0}{\gamma_T r_0^3} = \frac{300{,}000}{120 \times 717} = 3.5$$

FROM FIG. 16-4: $a_0 = 1.4$ (SOLID LINES)

$$f_0 = \frac{a_0}{2\pi r_0}\sqrt{\frac{Gg}{\gamma_T}} = \frac{1.4}{6.28 \times 8.95}\sqrt{\frac{3700 \times 32.2 \times 144}{120}}$$

$$= 9.43 \text{ CPS} = 566 \text{ RPM} < \frac{1250}{2} \text{ OK}$$

2. MAXIMUM AMPLITUDE AT RESONANCE

GREATEST VERTICAL FORCE OCCURS AT $f_0 = 566$ RPM

$$F_{f_0} = W_1\left(\frac{f_0}{f_1}\right)^2 = 4{,}000\left(\frac{566}{1250}\right)^2 = 820 \text{ LB}$$

$F_f = 2m_1 \ell \omega^2; \quad \omega = 2\pi f_0 = 6.28 \times 9.43 = 59.3$ CPS

$820 = 2m_1 \ell\, 59.3^2; \quad 2m_1\ell = .235$ LB SEC^2

FROM FIG. 16-1:

$A_{MAX}^{(1)}$ = Approx = .28 FOR b = 3.5 AND a_0 = 1.4

$$A_{MAX} = \frac{2m_1\ell}{\rho\, r_0^3} A_{MAX}^{(1)} = \frac{.235 \times .28 \times 32.2}{120 \times 717}$$

$= 0.0000247$ FT $= 0.0003$ IN < 0.0004 OK.

CONSTANT FORCE OSCILLATOR (CASE II).

GIVEN A VERTICAL-ACTING HAMMER WITH THE FOLLOWING CHARACTERISTICS:

$W_0 = 160{,}000$ LB; $\quad Q_1 = 2{,}000$ LB

$f_1 = 200$ RPM $\qquad A_F = 16' \times 8' \times 128$ SQ FT

FOUNDATION SOIL:

$\gamma_T = 120$ PCF $\qquad E = 8{,}000$ PSI

$\mu = 0.42 \qquad G = 2{,}820$ PSI

FIND:

1. RESONANT FREQUENCY
2. AMPLITUDE OF THE FORCED OSCILLATION.

1. RESONANT FREQUENCY

$r_0 = \sqrt{\dfrac{A_F}{\pi}} = \sqrt{\dfrac{128}{3.14}} = 6.38' \qquad r_0^3 = 260\ FT^3$

$$b = \frac{W_0}{\gamma r_0^3} = \frac{160{,}000}{120 \times 260} = 5.1 \text{ FT.}$$

FROM FIG. 16-4: $a_0 = 0.87$ (DASH LINES)

$$f_0 = \frac{a_0}{2\pi r_0}\sqrt{\frac{Gg}{\gamma_T}} = \frac{.87}{6.28 \times 6.4} \times \sqrt{\frac{2{,}820 \times 32.17 \times 144}{120}}$$

$= 7.15$ CPS $= 428$ RPM

$f_0/f_1 = \dfrac{428}{200} > 2$ OK

2. MAXIMUM AMPLITUDE

FROM FIG. 16-4, FOR b = 5.1 (DASH LINES) $A_{MAX}^{(2)} = 0.22$

$$A_{MAX} = \frac{Q_1}{G r_0} A_{MAX}^{(2)} = \frac{2{,}000 \times 0.22}{2{,}820 \times 6.4 \times 12} = .0021 \text{ IN.}$$

3. PLOTTING $A_{MAX} = 0.0021$ AND $f_1 = 200$ RPM ON UPPER DIAGRAM OF FIG. 16-1, INDICATES VIBRATION IS EASILY NOTICEABLE TO PERSONS.

THEREFORE FOUNDATION IS SUITABLE AS GIVEN BECAUSE OPERATING FREQUENCY IS LESS THAN HALF RESONANT FREQUENCY, AND ALTHOUGH THE VIBRATIONS ARE EASILY NOTICEABLE TO PERSONS THEY ARE NOT DAMAGING.

FIGURE 16-3
Example of Foundation Design to Avoid Resonance

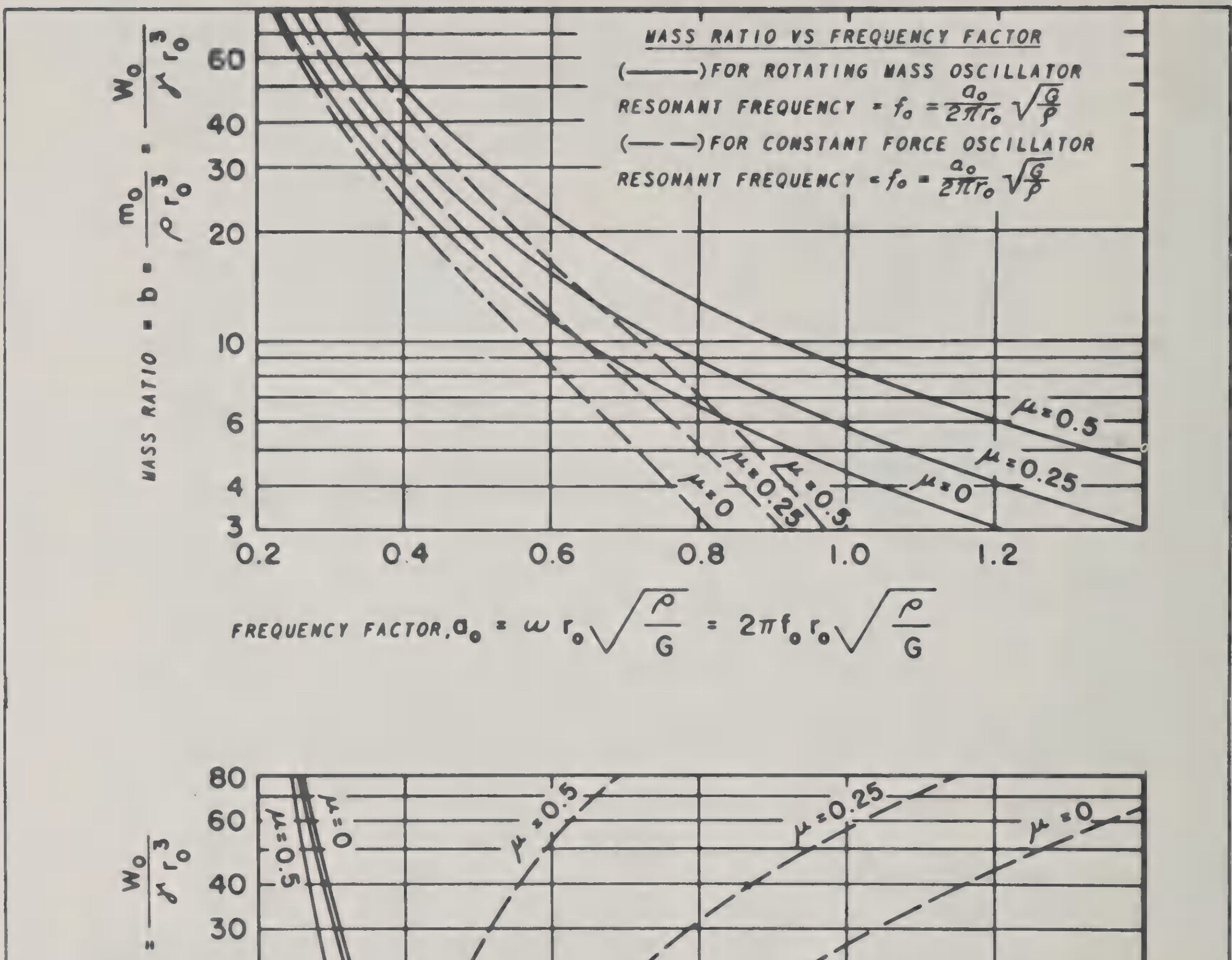


FIGURE 16-4
Relation between Mass, Frequency, and Amplitude of Oscillator

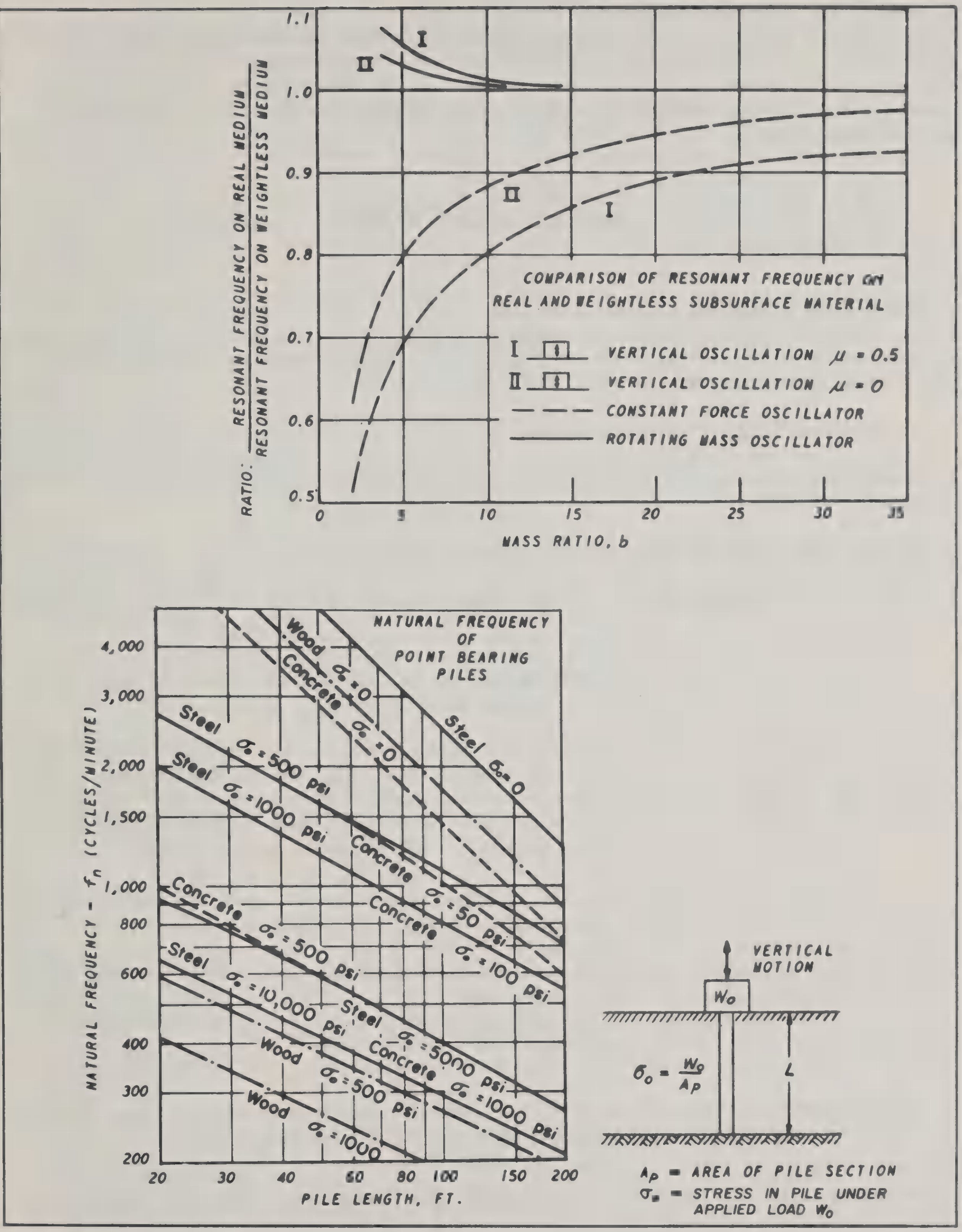


FIGURE 16-5
Values of Resonant Frequency

4. VIBRATION AND SHOCK ISOLATION. For general methods of isolating vibrating equipment or insulating a structure from vibration transmission, see Table 16-1. These methods include physical separation of the vibrating unit from the structure, or interposition of an isolator between the vibrating equipment and foundation or between the structure foundation and an outside vibration source. Vibration isolating mediums include resilient materials such as metal springs, or pads of rubber, cork, felt, or lead and asbestos in combination.

Section 4. SEISMIC EFFECTS

1. EARTHQUAKE LOADING. Earthquake damage depends on type of structure and foundation and subsurface conditions. Earthquake load is approximated for design by applying lateral forces equal to a fraction (generally between 10 and 20 percent, or 30 percent for certain vulnerable structures) of dead load plus a portion of live load. See NAVDOCKS P-355 for detailed criteria.

a. Foundation Loads. When foundations have been designed for safety factors required in Chapters 11, 12, and 13, an overload by seismic forces up to 33 percent of the dead plus normal live load ordinarily is permitted without redesign. For foundations on loose, cohesionless soil, special consideration must be given to the possibility of settlement or loss of bearing capacity from earthquake shock.

b. Wall Loads. Add a horizontal component to active wedge weight plus permanent surcharge and determine wall pressures by static analysis of active failure wedge as shown in Chapter 10. Allowable stresses in walls or retaining structures are increased for transient shocks in accordance with NAVFAC DM-2.

2. INFLUENCE OF SUBSOIL CONDITIONS. Weaker and less rigid subsoils magnify the amplitude of seismic vibrations and, for the majority of structures, damage is increased on soft subsoils.

a. General Effects. Much of the damage to light buildings on soft or loose soil results from differential settlement of the surface caused by natural variability of subsoils. Maximum damage occurs on unconsolidated fills. In seismically active regions, every effort should be made to compact the fills used for support to high density. In saturated loose or medium compact, cohesionless soils, seismic shocks may produce shear strains that develop high pore pressures, greatly decreasing bearing capacity.

b. Foundation Type. Although few data are available, mat foundations appear to permit less damage than pile foundations because mats act more nearly as a single unit in response to shock waves. In seismically active regions, consider the following:

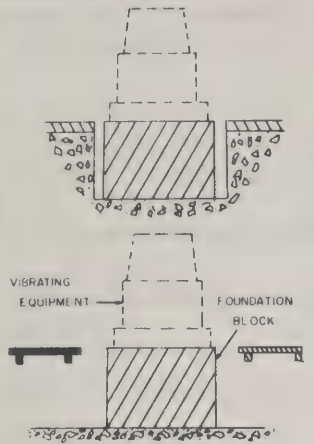
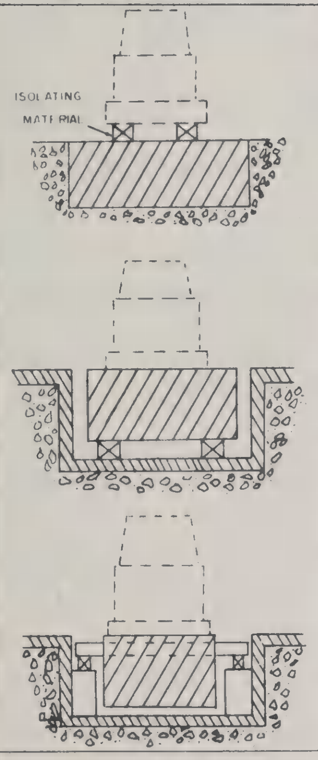
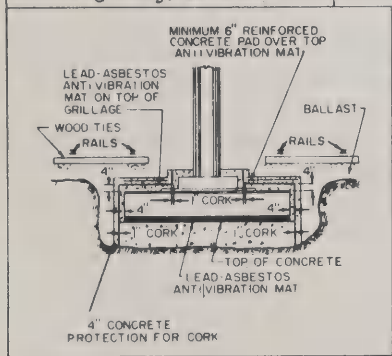
(1) If a pile foundation is necessary, piles should be battered at least in three directions and the pile foundation rigidly connected to the superstructure.

(2) If individual spread footings are used, connect the footings with reinforced concrete struts for stiffening.

3. EMBANKMENT AND SLOPE STABILITY. For analysis of seismic effect, add a horizontal component to the body force vector equal to the estimated percentage of gravity acceleration produced.

a. Safety Factor. For cohesive soils the transient shear strength for a loading time of one-quarter second (typical period of seismic shock waves) is 1.5 to 3 times the laboratory strength for tests of half hour duration. For cohesive soils a reduction of safety factor to 1.2 or 1.15 usually is tolerable for seismic loading. For relatively impervious, saturated, cohesionless soils, the possibility of shear pore pressures developing under seismic shock must be considered.

TABLE 16-1
Vibration and Shock Isolation

<table>
<tr><th colspan="2">Method of isolation</th><th>Applications</th></tr>
<tr><td>
</td><td>Physical separation of vibrating equipment from the structure. Equipment is installed on a concrete block set in or on the ground not in contact with the surrounding floor or structure.</td><td>This is the cheapest and simplest method of isolating vibrating equipment but the least effective. Frequently used for mounting machine tools or similar equipment where moderate vibration transmitted to the structure is tolerable and no sensitive instruments are involved.</td></tr>
<tr><td>
</td><td>Vibrating equipment is mounted with or without the base block, supported or surrounded on isolating material. Isolators consist of metal springs, rubber, cork or felt blocks, pads or mats, or other resilient material. Isolator may placed directly beneath the equipment or under a base block that provides additional inertia mass. In some cases, isolators are located at the center of gravity of the combined mass of equipment plus base block. In other cases the equipment is placed on a block suspended as a pendulum that is supported on the isolating material.</td><td>Isolators are utilized for a variety of vibrating equipment. Rubber isolators are frequently employed for engines and compressors. Heavy hammers and presses may be placed in pits lined by isolating material or supported on springs. Sensitive instruments frequently are mounted on isolating materials.</td></tr>
<tr><td>
</td><td>For insulation of structural frame against vibrations transmitted from the outside, insulating pads may be incorporated in the footings. One suitable method utilizes fabricated pads of lead and asbestos. In special circumstances it may be practical to isolate an entire building by surrounding it by a ditch. This should be at least 12 feet deep without cross bracing between sides. Should cross bracing be necessary it should be arranged with vibration isolating material incorporated in the bracing.</td><td>Lead-asbestos pads have been utilized for heavy individual column footings to reduce the vibration from railroad nearby. The pads are effective because vibrations resist being transmitted through dissimilar materials in contact.</td></tr>
</table>

b. Effect of Material Type. The above method of equivalent static analysis of slope stability is not entirely satisfactory to represent seismic effects. Observations indicate that the response of various materials to seismic shock generally follows the following patterns:

(1) Very steep slopes of weak, fractured, and brittle rocks or unsaturated loess are vulnerable to transient shocks because of opening of tension cracks, which overload the toe of slope.

(2) Loose, saturated sand or mountain detritus may be liquified by shocks with sudden collapse of structure and flow slides. Similar effects are possible in sensitive cohesive soils with natural moisture exceeding the liquid limit.

(3) Dry cohesionless material on a slope at the angle of repose will respond to seismic shock by shallow sloughing and slight flattening of the slope.

(4) Well-compacted cohesionless embankments or reasonably flat slopes in insensitive clays, which are safe under static conditions, will not fail in shear by seismic shocks. Cracking of the embankment is a dangerous possibility. For earth dams in earthquake regions, provide internal drainage of embankments to counteract cracking of the core by seismic shocks.

(5) A clay foundation beneath loose sand embankments may be overstressed if seismic shocks cause a significant loss of strength in the embankment interior.

BIBLIOGRAPHY

Barkan, D. D., *Dynamics of Bases & Foundations* (translated from Russian), McGraw-Hill Book Co., N.Y. 1962.

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

Foundations Subject to Vibratory Loads, U.S. Army Office Chief of Engineers, EM 1110-345-310.

Handbook of Drainage and Construction Products, Armco Drainage and Metal Products, Inc., Middletown, Ohio.

Hetenyi, M., *Beams on Elastic Foundation*, University of Michigan Press, Ann Arbor, Michigan.

Hvorslev, M. J., *Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes*, Department of the Army, Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss.

Jaeger, J. C. and Cook, N.G.W., *Fundamentals of Rock Mechanics*, Methuen & Co. Ltd., London, 1969.

Ketchum, M. S., *The Design of Walls, Bins and Grain Elevators*, Engineering News Publishing Co., New York, N. Y.

Mehta, M. R., and Veletsos, A. S., *Stresses and Displacements in Layered Systems*, University of Illinois, Structural Research Series No. 178.

Obert, L., and Duvall, W. I., and Merrill, R. H., *Design of Underground Openings in Competent Rock*, Bulletin 587, U. S. Bureau of Mines.

Soil Mechanics Design, Seepage Control, Chapter 1, Part CXIX, Engineering Manual, Civil Works Construction, Department of the Army, Corps of Engineers.

Stagg, K. G. and Zienkiewicz, O. C., *Rock Mechanics in Engineering Practice*, John Wiley & Sons Ltd., London, 1968.

Taylor, D. W., *Pressure Distribution Studies on Soils*, Soil Mechanics Fact Finding Study Progress Report, Department of the Army, Corps of Engineers, Waterways Experiment Station, Vicksburg, Miss.

Terzaghi, K., *Theoretical Soil Mechanics*, John Wiley & Sons, Inc., New York, N. Y.

Terzaghi, K., *Rock Defects and Loads on Tunnel Supports, Section 1 of Rock Tunnelling with Steel Supports*, The Commercial Shearing and Stamping Co., Youngstown, Ohio.

Van der Veen, C., and Boersma, L., *The Bearing Capacity of a Pile Pre-Determined by a Cone Penetration Test*, Proceedings, Fourth International Conference on Soil Mechanics and Foundation Engineering, Butterworths Scientific Publications, London.

Woods, K. B., *Highway Engineering Handbook*, McGraw-Hill Book Company, Inc., New York, N. Y.

Zanger, C. N., *Theory and Problems of Water Percolation*, Engineering Monograph No. 8, U. S. Bureau of Reclamation.

CRITERIA SOURCES

(Publications Containing Criteria Cited in this Text)

Abbett, R. W., *American Civil Engineering Practice,* Vols. I, II, III, John Wiley & Sons, Inc., New York, N. Y.

Arctic and Subarctic Construction, Engineering Manual TM-5-852-6, Department of the Army, Corps of Engineers.

ASTM Standards, Parts 9, 10, 11, & 12, American Society for Testing Materials, Philadelphia, Pa.

Chellis, R. D., *Pile Foundations—Theory, Design and Practice,* McGraw-Hill Book Company, Inc., New York, N. Y.

Earth Manual, U. S. Bureau of Reclamation, Denver, Colo.

Lambe, T. W., *Soil Testing for Engineers*, John Wiley & Sons, Inc., New York, N. Y.

Nishida, Y., *Brief Note on Compression Index of Soil,* Proceedings of the American Society of Civil Engineers, Volume 82, SM3, February 1956, American Society of Civil Engineers, New York, N.Y.

Procedures for Testing Soils, American Society for Testing Materials, Philadelphia, Pa.

Sanitary and Storm Sewers, ASCE Manual of Engineering Practice, No. 37, American Society of Civil Engineers, New York, N. Y.

Seelye, E. E., *Foundations, Design and Practice*, John Wiley & Sons, Inc., New York, N. Y.

Standard Specifications for Highway Materials and Methods of Sampling and Testing, Part III, American Association of State Highway Officials, Washington, D. C.

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

GLOSSARY

Activity of Clay. The ratio of plasticity index to percent by weight of the total sample that is smaller 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.

Argillaceous Rock. A sedimentary rock in which clay minerals or low grade micas (chlorite, sericite) predominate; e.g., shale, slate, argillite, or claystone.

Base Exchange. The physicochemical process in which one species of ions on the lattice of a clay particle are replaced by another species, thereby altering the plasticity and physical properties of the clay.

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.

Depth Creep. Continuous gradual movement of a slope occurring at a substantial depth under the influence of gravity forces. This is distinguished from seasonal creep, which occurs at the ground surface from thermal expansion and contraction, swelling, shrinkage, freezing and thawing, and other seasonal processes.

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.

Driving Resistance. The number of blows of a pile driving hammer required to advance the point of a pile driving hammer required to advance the point of a pile a specific distance into the subsoils.

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.

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.

Homogeneous Earth Dam. An earth dam whose embankment is formed of one soil type with out a systematic zoning of fill materials.

Hydrostatic 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.

Hydrostatic Pore Pressures. Pore water pressures or ground water pressures exerted under conditions of no flow where the magnitude of pore pressures increase linearly with depth below the ground surface.

Liquefaction. The sudden, large decrease of shearing resistance of a cohesionless soil caused by collapse of the soil structure, produced by shock or small shear strains, associated with sudden but temporary increase of pore water pressures.

Nominal Bearing Pressures. Allowable bearing pressures for spread foundation on various soil types, derived from experience and general usage, which povide safety against shear failure or excessive settlement.

Normal Consolidation. The condition that exists if a soil deposit has never been subjected to an effective pressure greater than the existing overburden pressure and if the deposit is completely consolidated under the existing overburden pressure.

Penetration Resistance. The number of blows of a specified weight hammer, falling a given distance, required to produce a specific penetration of a sampling or penetration device into subsoils.

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 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.

Rippability. The characteristic of dense and rocky soils that can be excavated without blasting after ripping with a rock rake or ripper.

Slickensides. Surfaces within a soil mass which have been smoothed and striated by shear movements on these surfaces.

Undercut. An excavation made to remove potentially swelling soils from beneath a foundation. The excavation is carried to a depth at which the magnitude of swell will be tolerable and the soils removed are replaced by nonswelling material.

Underconsolidation. The condition that exists if a soil deposit is not fully consolidated under the existing overburden pressure and hydrostatic excess 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.

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.

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