SOILS AND FOUNDATIONS FOR ARCHITECTS AND ENGINEERS

Second Edition

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by

Chester I. Duncan, Jr. FASCE
Professor of Architecture
University of Texas at Arlington

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To the following graduate students from
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Artwork by
Daniel Keith Maple
and
Sherry Leigh Penley

Editing by
Erin Marie Walters

Pictorial contributions by
Turgut M. Basdemir
Matthieu O. Chesaux
Matthew R. Farrington
James J. Kunz
Randy P. Mao
Marc A. McCollom
Kimi Sue Mittleman
Elizabeth A. Pekinpaugh
John D. Seeley
David B. Williamson

Foreword

Most of the time we only see half a building—the part above ground, the part that we creatures of the planet's skin can see as we move about on our horizontal plane. But this is a very naive view, somewhat like thinking that trees are leafy sticks somehow balanced on a point. In reality, trees have an almost mirror-image structure below the ground to support the visible part above. A wide system of underground branches we call roots collect and distribute the forces that keeps the tree erect and stable. This is also true of buildings. There is an elaborate and elegant system of construction below the ground—not visible—that makes possible the portion above.

The design and engineering that take place below grade are often monumental—and beautiful. Anyone who has peeked through the slats at a big building site and has seen the hole filled with other holes being filled with concrete and driven with steel will feel the power of what goes on beneath the surface of major construction.

Chester Duncan's book is a fine and much needed guide to this world of construction beneath the surface, to the places that anchor, support, and stabilize the structures that make up our built world. It takes us beyond the naive view of buildings sitting on the landscape and into the complex interlocking of the man-made and the earth that is the reality of what we construct.

This book also shows us how the engineering and design of buildings meeting the ground is empirical, based on the pragmatics and deep knowledge of a site. While architecture and engineering above ground can try to demonstrate the fact and fiction of ideal systems, meeting a highly composite and variable mix of materials like the earth's skin is a different matter. This layer we live on and in is the result of billions of years of mixing, folding, grinding, and compression. It is anything but the consistent and uniform substance that simple theories prefer. So the engineer and designer of foundations must bring a pragmatic eye and mind to the task, an ability to see and adjust to inconsistent and often puzzling conditions. Experience and the judgment that a good mind draws from it are required at every step. The more one has seen and heard about, the more one knows. Professor Duncan's book offers the guidance of almost a half-century of experience in the engineering of buildings. In this second edition of what has become a recognized designer's companion, Professor Duncan has added to his rich store of concepts, facts, and the illustrations through which we can better understand them. His experience is shared with the student and practitioner in a direct way. Most architects and engineers I know are more than a little apprehensive when it comes to dealing with soils and foundations—there are just too many unknowns and pitfalls, literally. They need a guide who knows the terrain and has seen what can and what ought to happen. Chester Duncan has written this sort of guide through the vagaries and circumstance of fixing our buildings to the land.

Edward M. Baum, AIA
Dean, School of Architecture
University of Texas at Arlington

Preface to the Second Edition

This book was written as a text for architectural and engineering students in undergraduate and graduate-level programs and as a reference for architectural and engineering practitioners, general contractors, and residential builders. It can also be used to provide attorneys and insurance adjusters with a fundamental understanding of subject matter upon which they may be called to litigate.

Through this book the student will learn the basic theory relating to soils and foundations, along with the application of that theory in practice. The practitioner, having already learned the basics, can use this text to provide guidance for field situations with which he or she may not be familiar. In addition to basic theory, this book introduces a variety of topics not normally found in the main stream of soils engineering literature, such as buoyancy, prestressed tribacks and tie-downs, the installation and design of rock anchors and residential concerns regarding expansive soil.

The theory of soil mechanics is based on the assumption that the soil in question is homogeneous and isotropic throughout the mass. Such ideal conditions are seldom realized in the field. The application of theory must therefore be tempered with judgment, and judgment can only come from hands-on experience. The author has included numerous recommendations based on hands-on-experience accumulated in 25 years of professional practice. These recommendations are aimed at adding valuable insight for those involved in the resolution of problems arising in design or in the field.

It has been said that a picture is worth a thousand words. Because architects and engineers are visually oriented, this book incorporates more than 230 illustrations, each of which tells a story aimed at adding a richer, more detailed understanding of the text.

It has been the author's intent to present the information in this book using a less formal style than that found in most textbooks. Although the term user-friendly has been overworked in recent years, this book was written with that style of writing in mind. For example, in areas in which the author calls on his experience or presents his recommendations the text takes on a somewhat conversational tone. It is sincerely hoped that the format used in writing this book succeeds in presenting the material in such a way as to be readily understood and interesting to read.

Suggestions and criticisms of readers are welcome. All will be appreciated and given thoughtful consideration.

Chester Duncan
Arlington, Texas

List of Symbols

The following is a list of symbols used throughout this text. These symbols are in general use in the soils and foundations industry. On occasion a symbol may be used to express two different things. Because the different use of these symbols occurs only in different topics, there should be no confusion as to the intent of a given symbol within a given topic. The units given in the definition are those most customarily used.

AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ASTM	American Society for Testing and Materials
A	area (si, sf)
a, w, v, s	subscripts, air, water, voids, solids, used in the analysis or testing of a soil sample
a_1, a_2	shape factors used in determining the ultimate bearing strength of a rectangular footing
B	the side or width of a footing (feet)
b, h	width and depth of shear keys (inches)
C_1, C_3	influence coefficients used to determine the pressure induced at various points in a soil mass by a concentrated or uniformly distributed load
C_c, C_u	the coefficients of curvature and uniformity, used in the identification of well graded and poorly graded coarse grained soils
C_N	a correction factor used to modify the blow count N due to the release of overburden
C_w	a correction factor used to modify the blow count N due to the existence of a water table
c	unit cohesion (psf)
D	diameter of a round footing (feet)

D_{10}, etc.	the particle diameter corresponding to the percent passing line on a particle distribution curve as identified by the subscript (mm)
D_c	the critical depth below which the overburden pressure effective in producing skin friction between sand and piles no longer increases with increased depth (feet)
D_f	the distance below grade to the bottom of a footing or to the point at which the shear resistance on the surface of a pile or pier is to be determined (feet)
D_r	the relative density of a coarse grained soil (%)
D_w	the distance to the water table measured from grade (feet)
e	the void ratio of a coarse grained soil, with various subscripts identifying the condition of the soil under which the void ratio is required
f	the ultimate shear capacity developed between the shaft of a pile or pier and the surrounding soil (psf)
f_b	allowable bending stress on an uncracked concrete section (psi)
f_{brg}	allowable bearing stress on concrete (psi)
f'_c	specified concrete strength in 28 days (psi)
f_r	modulus of rupture of concrete (psi)
G_s	specific gravity of the solid constituents of a soil
H, h	general expression for height (feet, inches)
h_s	height of solids in a consolidation test (inches)
i	the angle of rupture, measured from the horizontal (degrees)—Article 9-2 and Figure 9-5
K	the coefficient of lateral pressure used in the determination of the ultimate shear capacity between the shaft of a pile or pier and the surrounding soil
K_a	the coefficient of active pressure used in the determination of the lateral force P_a
K_h	the coefficient of horizontal pressure used with the charts of [Ref. 20]
K_v	the coefficient of vertical shear resistance used with the charts of [Ref. 20]
k	the coefficient of permeability (cm/sec)
l	lap and development lengths of reinforcing bars, with various subscripts to identify the specific situation (inches)
LL	liquid limit, relating to a clay soil
N	blow count, as determined in the standard penetration test
N_c, N_q, N_γ	bearing capacity factors, used in determining footing capacity
n	soil porosity
P	a concentrated load (kips, tons)
P_a	the lateral force exerted on a wall by the restrained earth wedge (pounds, kips)
$p_{1,2,3}$	the pressures occurring during a triaxial compression test (psf)

p	unit pressure, the location of interest being identified by various subscripts (psf)
p_f	resultant pressure used in settlement calculations (psf)
p_0	the pressure due to overburden (psf, tsf)
PI	plasticity index, relating to a clay soil
PL	plastic limit, relating to a clay soil
Q	a concentrated load used in determining the pressures within a soil mass (kips)
Q_{ult}	the ultimate load carrying capacity of a deep foundation (tons)
q	a uniformly distributed load used in determining the pressures within a soil mass (psf, ksf, tsf)
q_a	the allowable bearing capacity of a footing (psf, ksf, tsf)
q_d	the ultimate bearing capacity of a footing (psf, ksf, tsf)
q_u	the unconfined compression strength of soil or rock (psf, tsf)
R	the radius of a round footing (feet)
RQD	rock quality designation
S	the degree of saturation of a soil (%)
SF	safety factor
SL	shrinkage limit, relating to a clay soil
S_t	a measurement of sensitivity of a predominantly clay soil
s	the unit resistance to shear, the induced shear stress (psf)
USCS	Unified System of Soil Classification
USDA	United States Department of Agriculture
V	the volume of a soil or of its constituents, as identified by various subscripts (cf)
v	unit shear stress (psf, psi)
W	the weight of a soil or of its constituents, as identified by various subscripts (pounds)
w	water content, synonymous with moisture content (%)
α (alpha)	the angle of rupture, measured from the vertical (degrees)—Article 9-2 and Figure 9-5
Δ (Delta)	settlement of a spread footing (inches) or the increment of some finite thing
γ (gamma)	density, synonymous with unit weight (pcf)—Article 2-2
λ (lambda)	a cohesion reduction factor used to determine the ultimate shear capacity between a pile or pier and the surrounding clay—Article 8-10
ϕ (phi)	the angle of internal friction of a granular or mixed-grained soil (degrees)—Articles 2-12 and 9-2
Θ (Theta)	the angle of repose of an unrestrained soil mass (degrees)—Article 9-2
$\tan \delta$ (delta)	the coefficient of friction between the shaft of a pile or and the surrounding soil—Article 8-11

British–Metric Conversions

	British	Metric Equivalent
Length	inches	2.54 centimeters (cm)
	feet	0.3048 meters (m)
Area	square inches	6.452 cm^2
	square feet	0.0929 m^2
Force	pounds	0.4535 kilograms (kg)
	pounds	4.448 newtons (N)
	kips	4.448 kilonewtons (kN)
	tons	8.896 kN
Density	pcf	16.02 kg/m^3
Stress	psi	0.6895 N/cm^2
	psi	6.895 kilopascals (kPa)
	psf	47.88 N/m^2 = 47.88 Pascals (Pa)
	ksf	47.88 kPa
	tsf	95.76 kPa
Moments	inch kips	11.52 kg m
	foot kips	1.356 kN m

Note: 1N = 0.1019 kg = 0.2248 pounds
1 Pa = 1 N/m^2 = 0.0209 psf
1 MPa = 1000 kPa = 145 psi

1

Classification of Soils

1-1. INTRODUCTION

The surface of the Earth contains many different kinds of soil, each having its own unique physical and chemical characteristics. The forces acting on a building are transmitted through the foundations into the ground, where the underlying soil provides the ultimate response. Each kind of soil exhibits a different kind of response. The design of the foundations cannot be completed, nor should it even be started, until the behavior of the underlying soil has been determined.

The first step in determining the behavior of the different kinds of soil is to classify them according to certain common physical and chemical characteristics.

1-2. SOIL TERMINOLOGY

In order to begin a study leading to the classification of soils and an understanding of soil behavior, it is first necessary to understand the meaning of the terms used to describe the various kinds of soil.

Rock

Rock is a naturally formed material composed of mineral grains connected together by strong, permanent cohesive forces to form a solid, impervious mass having some degree of mineralogic and chemical consistency.

Although rock is not a soil, rock is the origin of all soils. When rock is subjected to the physical or chemical action of air or water for extended periods of time, the rock will disintegrate and decompose. This phenomenon of change is commonly called weathering. All soils are the end product of the weathering

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of rock. The characteristics of rock, as related to building construction, are discussed in Chapter 14.

Soil

All soils are a natural aggregate of mineral grains which can be separated by gentle agitation in water. Soil grains are separated by size into four general classifications: gravel, sand, silt, and clay. Gravel and sand are referred to as coarse grained soils, while silt and clay are referred to as fine grained soils. In their natural state soil masses are rarely homogeneous and contain both coarse and fine grained fractions. Such soils are referred to as mixed grained.

Coarse Grained Soils

Coarse grained soils are defined as those soils whose individual grains are retained during the sieve test on sieves larger than, and including, the No. 200 (0.075-mm) sieve. Grains of this size can generally be seen with the naked eye, although a hand-held magnifying glass may occasionally be needed to see the smallest of the grains. Gravel and sand are coarse grained soils.

When a coarse grained soil has been oven dried it can easily be separated into individual grains. The presence of moisture does not affect the separation of the larger grains but will impart a degree of stickiness to the smaller particles (referred to as the finer fractions) of sand. Soils exhibiting predominantly coarse grained characteristics are sometimes referred to as granular soils.

Fine Grained Soils

Fine grained soils are defined as those whose individual particles pass a No. 200 sieve. Particles of this size can usually not be seen with the naked eye, even with the aid of a magnifying glass. Examination of these particles, therefore, must be made with optical and electron microscopes. Silt and clay are fine grained soils. All fine grained soils exhibit, to some degree, the properties of plasticity and cohesion. Soils exhibiting predominantly fine grained characteristics are sometimes referred to as cohesive soils.

Mixed Grained Soils

Soils containing measurable amounts of coarse and fine grained fractions are referred to as mixed grained soils. The properties and characteristics of a mixed grained soil depend to a large extent on the distribution between the two fractions. When the fine grained fraction is approximately one-third of the mix, the soil will exhibit the general properties and characteristics of a fine grained soil.

Gravel

Gravel makes up the larger fraction of the coarse grained soils. Most gravels have a distinctly rounded shape and are smooth to the touch. Unlike sand, gravel exhibits no tendency to stick together when wetted.

Sand

Sand makes up the smaller fraction of the coarse grained soils. Sand, unlike gravel, exhibits considerable variation between grains. Grains can be described as round, angular, smooth, or sharp. Sand can easily be separated by gently shaking when dry, but the smaller grains exhibit a definite tendency to stick together when wet. Sand is subdivided into three classifications based on particle size—coarse, medium, and fine.

Silt

Silt makes up the coarser portion of the fine grained fraction of soils. Silt acts somewhat as a transition between sand and clay because it has some of the properties of each. Silt, like sand, consists of rock fragments that have not been chemically altered. The mineralogical composition of these grains, therefore, remains essentially that of the rocks from which they were derived. On the other hand, silt exhibits considerable plasticity and cohesion, which are properties of clay.

Silt is subdivided according to plasticity. The fraction with the least plasticity consists primarily of very fine rounded grains of quartz and is called rock flour. The fraction with the most plasticity consists primarily of flake-shaped particles and is called plastic silt.

Clay

Clay makes up the finer portion of the fine grained fraction of soils and is the end product of the chemical decomposition of rock. The mineralogy and molecular arrangement of a clay particle are extremely complex and highly variable. This gives rise to a considerable range of characteristics within the overall family of clays. Clays are subdivided, therefore, into several groups that differentiate one clay type from another.

Organic Soil

Organic soil is defined as any soil which contains decayed vegetable or animal matter in any amount, no matter how small. Soil containing organic matter can usually be identified by its rich brown color and distinctive odor. Organic soil is totally unsuitable as a material on which to build any part of a building structure or to use as backfill against basement or retaining walls.

Inorganic Soil

This term refers to any mixture of soil that is completely free of organic constituents. Only inorganic soils can be considered as potentially acceptable structural materials, subject to further analysis.

Loam

Loam is a loose textured mixture of sand, silt, and clay that can be easily worked with garden tools.

Top Soil

Top soil is a mixture of loam and organic material and is the very best of soil mixtures within which to grow plants and vegetation.

Peat

Peat is a highly organic, fibrous, and relatively light soil usually found in marshy areas. Because of its extreme compressibility, it is totally without merit as a structural soil.

Glacial Till

Glacial till is a heterogeneous mixture of clay, sand, pebbles, and boulders transported and then deposited by glaciers.

Hardpan

Hardpan is a cemented and compacted layer of extremely dense, impenetrable clay or glacial till.

1-3. SOIL CLASSIFICATION SYSTEMS

There are four major soil classification systems. These have been developed in order to standardize technical information from which the characteristics and load response of a given soil can be approximated for the purpose under which the soil is to be used.

1. United States Department of Agriculture (USDA)
2. American Association of State Highway and Transportation Officials (AASHTO)
3. Unified Soil Classification System (USCS)
4. American Society for Testing and Materials (ASTM)

The soils identified within these classifications are grouped in accordance with the following properties and characteristics:

1. Particle size
2. Particle distribution
3. Plasticity

Coarse grained soils are normally classified by particle size and the distribution by weight of the different sizes throughout the mass. Fine grained soils are normally classified according to plasticity. It should be emphasized that all these properties can only be determined by performing exacting tests in the controlled environment of a testing laboratory.

1-4. CLASSIFICATION BY PARTICLE SIZE

General

The size and distribution of particles in coarse grained and mixed grained soils are determined by performing a laboratory test called sieve analysis. The size and distribution of particles in a fine grained soil are usually not considered to be important. When required, however, they can be determined by performing a sedimentation test. Both the sieve analysis and the sedimentation test are described in the following ASTM Standard:

> ASTM Designation D-422: Standard Method for Particle-Size Analysis of Soils

This method provides for the quantitative determination of the size and distribution of particles within a soil mass. Soils retained on sieves down to and including a No. 200 sieve are classified as coarse grained soils. Soils that pass through a No. 200 sieve are classified as fine grained soils.

Before performing either of these two tests, the soil must be prepared in accordance with the following ASTM Standard:

> ASTM Designation D-421: Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants

This standard describes the procedure to be used in preparing the soil sample after it has been received from the field. In general it is required that the sample be exposed to the air at room temperature until it is thoroughly dried, after which any clumps of soil shall be broken up into separate grains with mortar and pestle.

Sieve Test

A standard sieve test, when performed on an oven dried, coarse grained sample, will provide the following information:

1. The identification of the soil as determined by particle size
2. The distribution of particle size within the mass

The sample must be oven dried in order to remove all moisture; otherwise, particles could stick together and give completely erroneous results.

This test is performed on a series of sieves, similar to those illustrated in

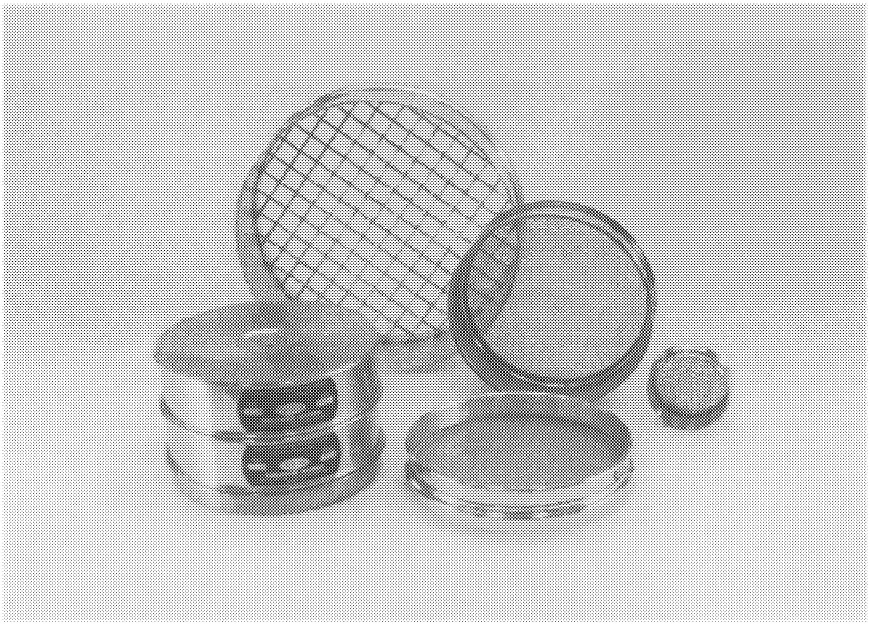


FIGURE 1-1. Standard sieves for particle size analysis. [Ref 4]

Figure 1-1. In order to perform the test, the sieves are vertically stacked in decending order of opening size. The prepared sample is placed on the uppermost sieve, and the stack is attached to a mechanical shaker which imparts an upward and sideways motion to the sieves. Each particle will fall through the stacked sieves until being retained on the sieve whose openings are smaller than the size of the particle. The larger particles are retained on the upper sieves, while the smaller particles are retained on sieves positioned lower in the stack. Sieves are manufactured to conform to the following ASTM Standard:

ASTM Designation E-11: Wire-Cloth Sieves for Testing Purposes

Sieves are manufactured in several different diameters. The 8 inch sieve, however, is the diameter most frequently used. Each sieve is identified by a number which identifies the size opening within the mesh. The most frequently used sieves are the 3-inch, 3/4-inch, and the Nos. 4, 10, 40, and 200. The clear width of the opening between the strands of mesh, as illustrated in Figure 1-2, is given in Table 1-1.

It should be noted that reference to particle size can be misleading because many of the particles are irregularly shaped. A particle, of course, is three-dimensional, and a particle may be larger than the sieve opening in one of its dimensions but smaller in the other two dimensions. Whether such a particle will pass through or be retained on a particular sieve depends on how the particle is aligned with respect to the sieve opening.

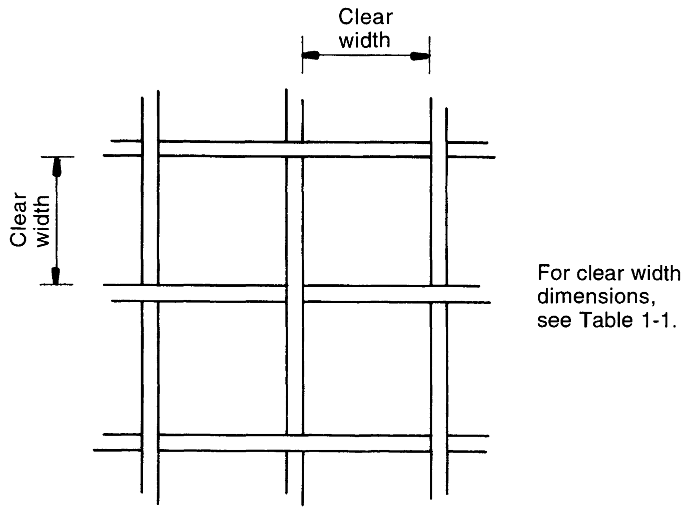


FIGURE 1-2. Sieve construction showing clear width of openings.

Particles are then classified by name according to the sieve upon which they were retained. This classification is given in Table 1-1.

Limitation to Classification by Particle Size

Soil classification by particle size is considered to be only the first step in determining the characteristics and load response of a given soil. The characteristics of a coarse grained soil depend not only on particle size but on shape,

TABLE 1-1. USCS Classification by Particle Size [Ref. 22]

Sieve Size/No.	mm[a]	in.[b]	Classification
			Cobbles
3″	75	3	
			Coarse gravel
3/4″	19	3/4	
			Fine gravel
No. 4	4.750	3/16	
			Coarse sand
No. 10	2.000	5/64	
			Medium sand
No. 40	0.425	1/64	
			Fine sand
No. 200	0.075	1/320	
			Silt or clay

[a] Dimensions are clear width of opening, see Figure 1-2.
[b] Sizes are approximate and are for general interest only.

distribution throughout the mass, and the arrangement of intergranular seating between particles. The characteristics of the fine grain fraction of soils—silts and clays—are primarily independent of particle size. If a particle of silt, for example, were reduced in size to that of a clay, the particle would not acquire the properties of clay. Nor do all clays possess the same properties. These properties are dramatically affected by the mineralogy and morphology of the particles. This is particularly true in the way in which clays of different mineralogy exhibit completely different characteristics in the absorption and release of pore water.

1-5. CLASSIFICATION BY PARTICLE DISTRIBUTION

General

In order to determine the distribution of particle size throughout the mass, the amount of sample contained on each sieve is carefully weighed. This weight is then converted to a percentage of the total weight of the original sample. The results of this test can be plotted on a particle distribution curve, an example of which is illustrated in Figure 1-3.

Different Kinds of Particle Distribution

The particle distribution curve can be a useful tool in identifying the uniformity or lack of uniformity with which the particles are distributed within the soil. Some soils contain grains representative of a wide range of particle size. Such

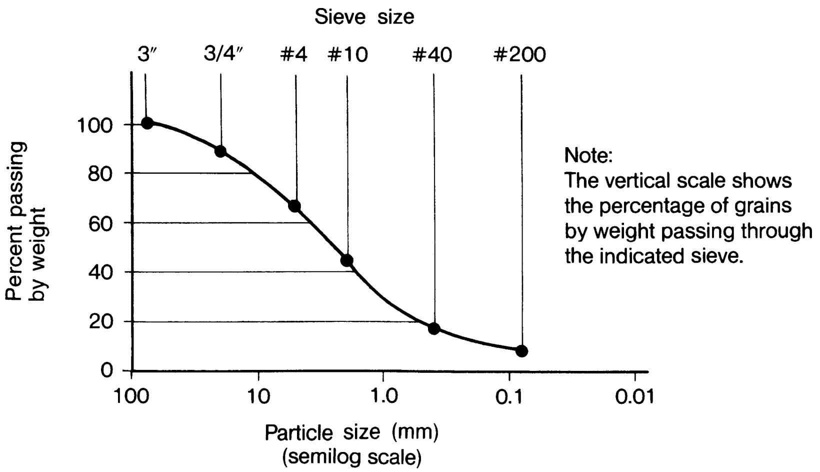


FIGURE 1-3. Typical particle distribution curve, as determined from sieve analysis.

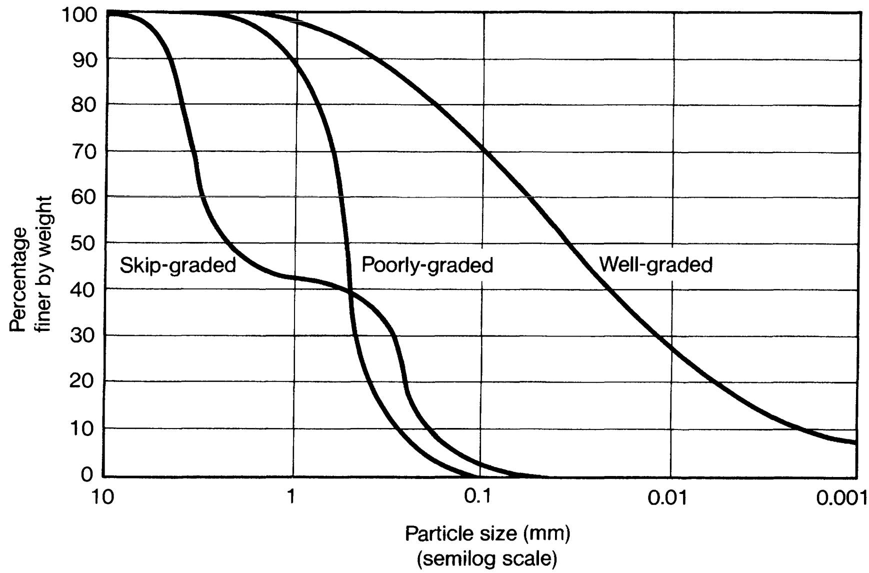


FIGURE 1-4. Particle distribution curves representative of well graded, poorly graded, and skip graded particles. [Ref 10]

soils are referred to as well graded. A soil whose grains are contained within a relatively small range of particle size is referred to as poorly graded. A soil whose grains extend over a wide range of sizes, but are lacking in the mid-range, is referred to as skip graded. Curves representative of these three different kinds of particle distribution are illustrated in Figure 1-4.

Coefficients of Uniformity and Curvature

When less than 5% of a soil identified as a sand or gravel passes a No. 200 sieve, it has been observed that there are not enough fines to significantly influence the behavior of the mix. The behavior of such a mix will be influenced primarily by grain distribution.

The coefficient of uniformity (C_u) and the coefficient of curvature (C_c) are used to determine whether a particular sand or gravel should be classified as well graded or poorly graded. In this analysis skip graded soils are classified as poorly graded. The numerical value of these coefficients is computed as follows:

$$C_u = \frac{D_{60}}{D_{10}} \tag{1-1}$$

$$C_c = \frac{D_{30}^2}{D_{60} \times D_{10}} \tag{1-2}$$

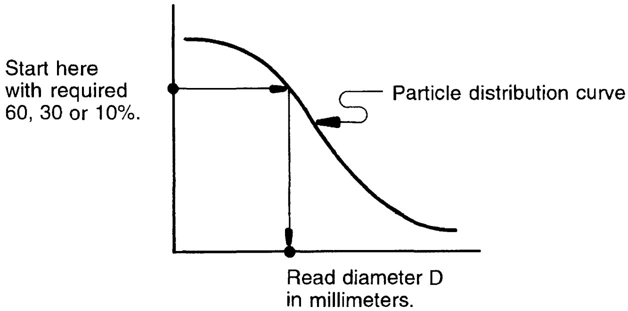


FIGURE 1-5. Procedure for determining D values used in computing the coefficients of uniformity and curvature.

In these formulas, D represents the diameter of the particle corresponding to the percent passing as specified numerically by the subscript. For example, D_{60} represents the particle size in millimeters corresponding to the percent passing as shown on the particle distribution chart. In order to determine the D values, it is first necessary to plot the particle distribution curve for the soil in question. D values may then be read from the curve, using the procedure shown in Figure 1-5 and demonstrated in Example 1-1.

The coefficient of uniformity will generally increase with the degree to which a soil is well graded. There is a problem of interpretation, however, because a skip graded soil may also demonstrate a relatively high value for this coefficient. The coefficient of curvature is used to overcome this problem. Soils that conform to the following limitations may be classified as well graded; those not conforming are classified as poorly graded:

$C_u > 4$	and	$3 > C_c > 1$	for gravel
$C_u > 6$	and	$3 > C_c > 1$	for sand

Sedimentation Test

Sedimentation tests, although to some degree approximate, are considered useful in determining the general size and distribution of particles within a fine grained soil. Such information is important to the architect and engineer because it gives valuable insight as to the variation in behavior that exists within the family of fine grained soils.

The sedimentation test is performed by preparing a soil solution in distilled water and allowing the soil grains to settle out without interruption.

This test is based on a theory known as Stoke's Law in which the diameter of an assumed spherical particle may be equated to the velocity with which it freely settles out of a fluid. During the test it is observed that the various grains of soil slowly settle out of the soil–water solution with the heavier grains settling out first. Required measurements are taken from which the general size and distribution of the particles can be determined. All work relative to this procedure shall be in accordance with ASTM Designation D-422, as specified in Article 1-4.

As previously noted, the results of this test are somewhat approximate. This is because the test computations are based on the assumption that the particles are spherical in shape and that all particles are of similar specific gravity. These may not be valid assumptions.

1-6. CLASSIFICATION BY PLASTICITY

Plasticity is the property of a soil that enables it to be remolded and deformed without separating or breaking apart. Plasticity plays an important part in the behavior of fine grained soils and in coarse grained soils containing a noticeable fine grained fraction.

All clays exhibit plasticity although not to an equal degree. Silts also exhibit plasticity, but to a considerably lesser degree. Plasticity is measured in terms of the plasticity index, which is a function of the water content of the soil in question. This index is one of the four boundaries of water content that comprise the Atterberg limits. These boundaries are identified as the liquid limit (*LL*), plastic limit (*PL*), shrinkage limit (*SL*), and plasticity index (*PI*). The meaning of these boundaries, and the procedures by which they are quantitatively obtained, are discussed in Article 13-6. The use of plasticity in the classification of soils is demonstrated in Article 1-9.

1-7. USDA CLASSIFICATION SYSTEM

The United States Department of Agriculture has developed a system of soil classification based solely on the distribution by weight of sand, silt, and clay throughout the soil mass. This is commonly referred to as the textural system of classification. It is an easy system to use and has the advantage of using terms with which all architects and engineers are familiar. To use this system, the percentages of dry weight of sand, silt and clay must be known. The intersection point of these three percentages is then located on the textural chart of Figure 1-6, and the name given to that particular mixture is read off of the chart. For example, a mixture consisting of 60% sand, 30% silt, and 10% clay would be identified as sandy loam.

The preceding method of classification cannot be used when the mix contains a substantial amount of gravel. For those instances when a small amount is present, the terminology should be modified to express that fact. If a small amount of gravel were present in the mixture of the previous example, that mixture would then be identified as a stony sandy loam or as a gravelly sandy loam.

One of the principal benefits of this system of identification is that it standardizes terminology in terms of conversational descriptions. When this chart is used

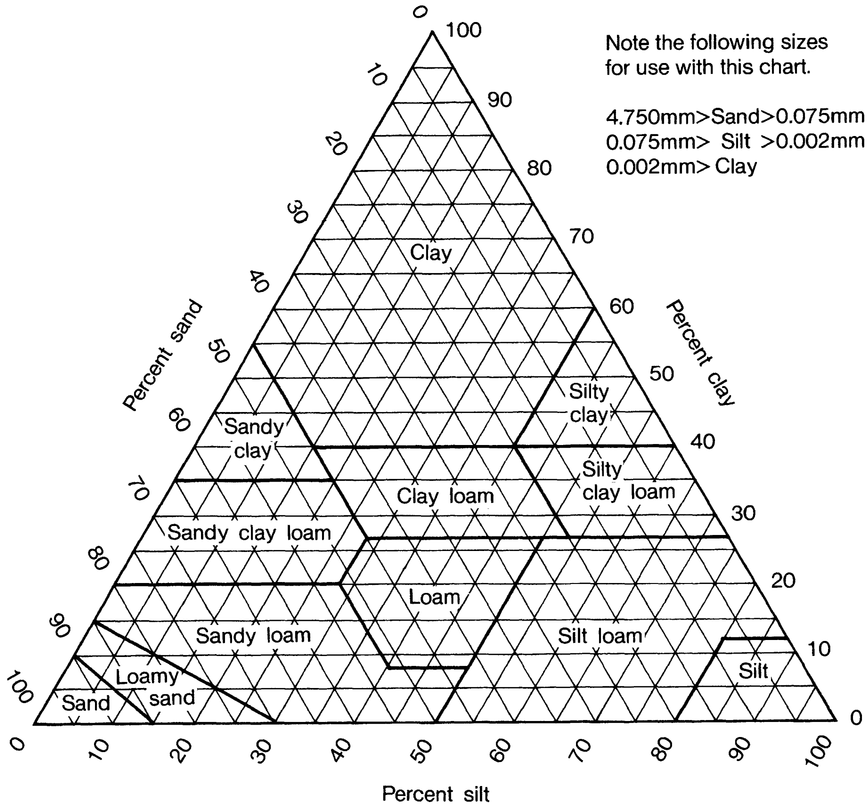


FIGURE 1-6. USDA textural classification of soils. [Ref 23]

correctly, each term has the same generalized meaning, no matter where it is used or by whom. This is a plus. The author has experienced numerous instances in which a term used to identify a particular soil had its roots more in local flavor than in the more widely accepted terminology.

The primary use of this system, of course, is agriculture. In the design of the foundations of a building (which is the thrust of this text), this system is of little use. In the landscaping of this building, however, this system has considerable merit.

The use of this relatively simple system of soil identification is complicated by the need to accurately know the percentages of each constituent. In serious engineering, however, this is just one of the many things that must be done.

1-8. AASHTO CLASSIFICATION SYSTEM

AASHTO Designation M-145, which originated with the Bureau of Public Roads, was ultimately adopted by the American Association of State Highway and

Transportation Officials for the purpose of identifying different kinds of soils in terms of their suitability for use in highway construction. This system classifies soils into eight groups, designated A-1 through A-8. Several of these groups are subdivided into smaller groups in order to further distinguish soils having different properties.

A soil consisting of well graded sand and gravel and containing a small amount of clay binder is considered the most suitable soil upon which to construct a highway. This soil is classified A-1. Other soils are classified in decreasing order of suitability. Organic soils, which are considered to be completely unsuitable, are classified A-8. The determination of suitability considers grain size, distribution, and plasticity.

Although considered a standard reference in highway construction, this system of classification is not generally used by architects and engineers engaged in the design of buildings.

1-9. THE UNIFIED SOIL CLASSIFICATION SYSTEM

General

The Unified System of Soil Classification is based on the results of extensive work performed in this area by Dr. Arthur Casagrande, Professor of Soil Mechanics and Foundation Engineering, Harvard University. The identification of soils according to this system is based not only on grain size and distribution, but on the behavior of the soil as characterized by plasticity as well.

This system, sometimes referred to as the USCS, has been adopted as standard by the Army Corps of Engineers and is the basis on which the ASTM Designation D-2487 was developed. The USCS system has gained acceptance throughout the industry and is now widely used by architects, engineers and contractors as the standard method of soil classification.

This system separates soils into fifteen groups, each of which is identified by a primary and secondary letter. The primary letter identifies the predominant soil type. The secondary letter provides additional information relating to the particular properties of the soil. Properties considered are the secondary soils within the mix, the grain distribution of those that are primarily coarse grained, and the characteristics of plasticity for those which are primarily fine grained. These letters and a description of their meaning are presented in Table 1-2.

The Fifteen Soil Classifications

The accurate identification of soils intended for structural use is of primary importance to the success of any project. This not only involves an understanding of how a particular soil will function in a particular situation, but also the need for a common terminology that can be understood by all those working on the

TABLE 1-2. USCS Primary and Secondary Designations [Ref. 22]

Primary Letter	Secondary Letter
G = Gravel	For coarse grained soils:
S = Sand	W = Well graded
M = Silt	P = Poorly graded
C = Clay	M = Coarse material with nonplastic fines or fines with low plasticity
O = Organic	C = Coarse material with plastic fines
PT = Peat	For fine grained soils:
	L = Relatively low liquid limit
	H = Relatively high liquid limit

project. All soils and mixtures of soils can be identified as belonging to one of the fifteen general soil groups of the Unified Soil Classification System. These groups, including a general description of each soil, are listed in Figure 1-7. The purpose of performing the various tests cited in this chapter is to determine into which of these 15 groups the soil should be placed.

To Distinguish Between Organic and Inorganic Soils

An important step in the process of soil identification is to determine whether the soil is primarily organic or inorganic. The existence of even a small amount of organic material voids that particular soil for use in any structural capacity, including subgrade for slab on ground, paving, roadwork, embankments, and backfill.

The existence of organic material in any given sample of soil may be determined by performing a variety of tests, including:

1. The presence of vegetable matter, such as sticks, leaves, or grass, imparts a typically fibrous texture to the soil.
2. The color of a moist organic soil will usually contain dark or drab shades of gray or brown and may include colors that are almost black.
3. The color of an inorganic soil, for comparison, contains brighter colors, including medium and light gray, olive green, brown, red, yellow, and white.
4. The odor of a fresh sample of an organic soil is distinctive, although it gradually diminishes on exposure to air. The original odor can be revived by heating a wet sample.
5. The moisture content (Article 2-5) of an organic soil may be as high as several hundred percent, which far exceeds that found in most soils.
6. The specific gravity (Article 2-6) of an organic soil is normally lower than that of an inorganic soil due to the presence of vegetable matter and a higher percentage of water and air.
7. For those few instances when there may be some doubt as to whether a soil is organic or inorganic, a laboratory test should be performed in order

Major Divisions			Symbol	General Description
Coarse Grained Soils, more than 50% of dry weight retained on a No. 200 sieve	Gravels, greater % of coarse fraction retained on a No. 4 sieve	Clean gravels	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels, gravel-sand-silt mixture
			GC	Clayey gravels, gravel-sand-clay mixture
	Sands, greater % of coarse fraction passes a No. 4 sieve	Clean sands	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		Sands with fines	SM	Silty sands, silt-sand mixtures
			SC	Clayey sands, sand-clay mixtures
Fine Grained Soils, 50% or less of dry weight retained on a No. 200 sieve	Silts and Clays, liquid limit 50% or less		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays, liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
Highly organic soils			PT	Peat and other highly organic soils

FIGURE 1-7. USCS General Soil groups. [Ref 22]

to compare the plasticity indices of a moist sample and an oven dry sample. The plasticity index of the oven dry sample will show a radical drop from that which was found in the moist sample. When the same test is performed on an inorganic soil, the plasticity index will vary only a few percentage points up or down.

Laboratory Identification of Soils by the USCS Flow Charts

The USCS groups all soils into one of two major divisions: (a) coarse grained, and (b) fine grained. The appropriate division into which a given soil should be placed can usually be made by visual examination. In borderline cases, or in cases of uncertainty due to the presence of considerable mixed grained material, a sieve analysis should be run on a representative, oven dried sample of soil. The purpose of this test is to determine the percentage of dry weight retained on a No. 200 sieve. Soils are classified as coarse grained when more than 50% of the dry weight is retained, and as fine grained when 50% or less is retained.

The classification of a coarse grained soil is generally dependent on particle size and distribution. In those instances when the soil contains a significant fraction of fines, plasticity must also be considered. The classification of a fine grained soil, on the other hand, depends solely on plasticity.

The classification of both types of soil is determined by the use of a flowchart that guides the examiner through a series of well organized steps. When plasticity must be considered the final classification of the soil will be determined from a plasticity chart. The charts required for each analysis are found in the following figures:

Major Division	Flowchart	Plasticity Chart
Coarse grained	Figure 1-8	Figure 1-11
Fine grained	Figure 1-9	Figure 1-10

The use of these charts is described in the following paragraphs.

Plasticity Chart for Fine Grained Soils

The plasticity chart shown in Figure 1-10 is used to determine the classification of fine grained soils. This classification is a function of the liquid limit (*LL*) and plastic limit (*PL*) of the soil. These limits, which are based on the water content of the soil, belong to the family of Atterberg limits, defined in Article 13-6.

Before using the plasticity chart, the liquid limit and plastic limit must be determined by laboratory analysis. The plasticity index (*PI*) is then computed as follows:

$$PI = LL - PL \text{ (all in \%)}$$

The chart is then entered with the known liquid limit and plasticity index. Identification is made depending on where these coordinates plot on the chart.

The primary purpose of the chart is to differentiate between clays, silts, and organic soils. Clays plot above the A line, while silts and organic soils plot below the A line. This line has been arbitrarily established according to the following formula:

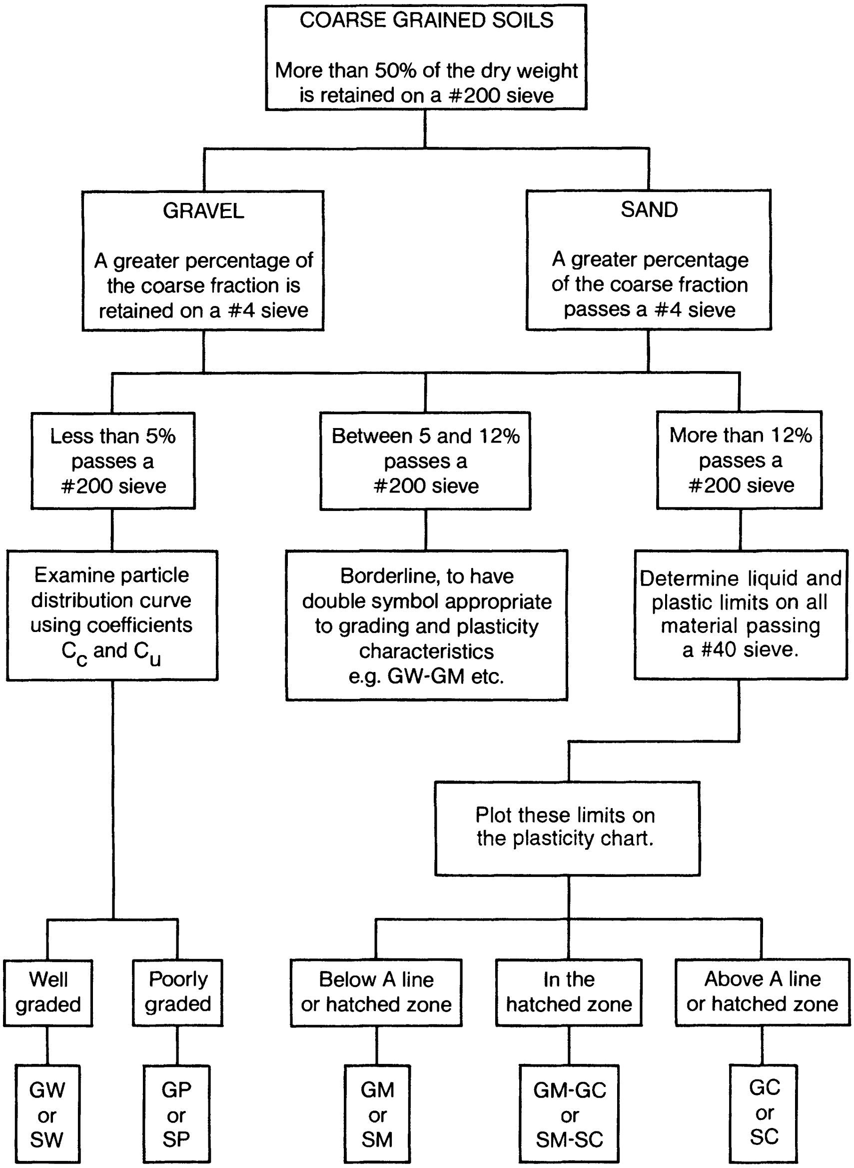


FIGURE 1-8. USCS Flowchart for identification of coarse grained soils. [Ref 22]

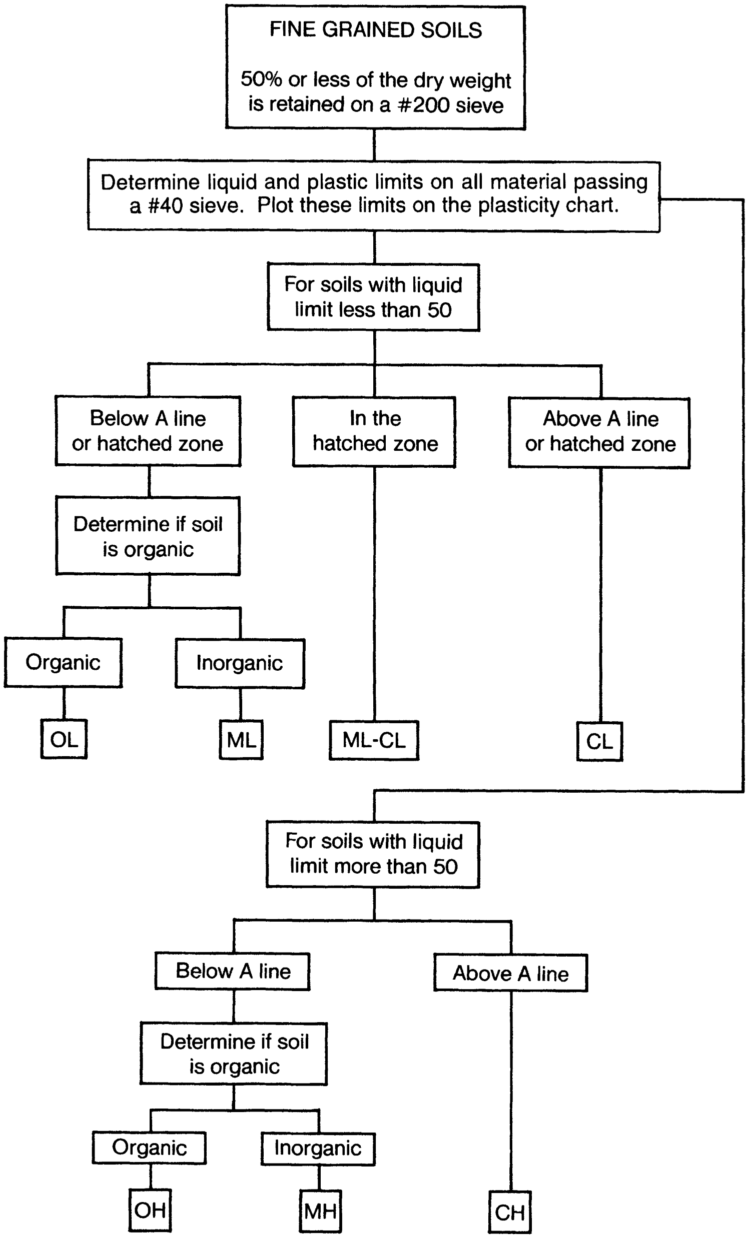


FIGURE 1-9. USCS Flowchart for identification of fine grained soils. [Ref 22]

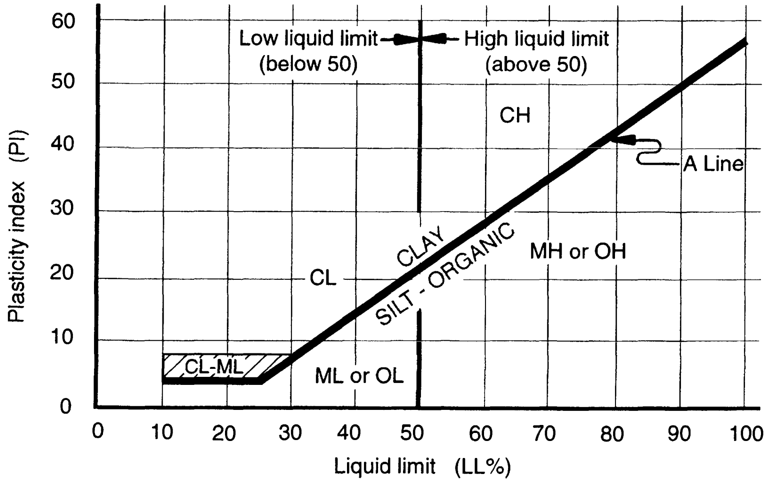


FIGURE 1-10. Plasticity chart, used in the identification of fine grained soils. [Ref 22]

$$PI = 0.73\ (LL - 20)$$

At a liquid limit below approximately 29, and at a plasticity index in the range of 4 to 7, there is an overlapping of characteristics between the *CL* and *ML* groups. This is the area which is shown hatched on the chart. Soils above this hatched area are *CL*, and soils below are *ML*. Those that fall within the hatched area are assigned the dual designation of *CL – ML*.

Plasticity Chart for Coarse Grained Soils

A secondary use of a plasticity chart is to differentiate between soil groups GM and GC, and soil groups SM and SC. These are the groups that have more than a 12% fraction passing through a No. 200 sieve. Because of their relatively high content of fines, it is necessary to determine whether these soils exhibit any marked degree of plasticity. In order to make this determination, the liquid and plastic limits are determined on the fraction of soil passing a No. 40 sieve. These limits should be determined on moist samples rather than on oven dry samples. By plotting the liquid limit and plasticity index of each sample on the plasticity chart shown in Figure 1-11 the proper soil identification can be made. Soils plotting below the A or the hatched zone are classified as GM or SM, those within the hatched zone are classified as GM-GC or SM-SC, and those above the A line or the hatched zone are classified as GC or SC.

1-10. ASTM CLASSIFICATION SYSTEM

In 1969 The American Society for Testing and Materials adopted the Unified Soil Classification System under the following title:

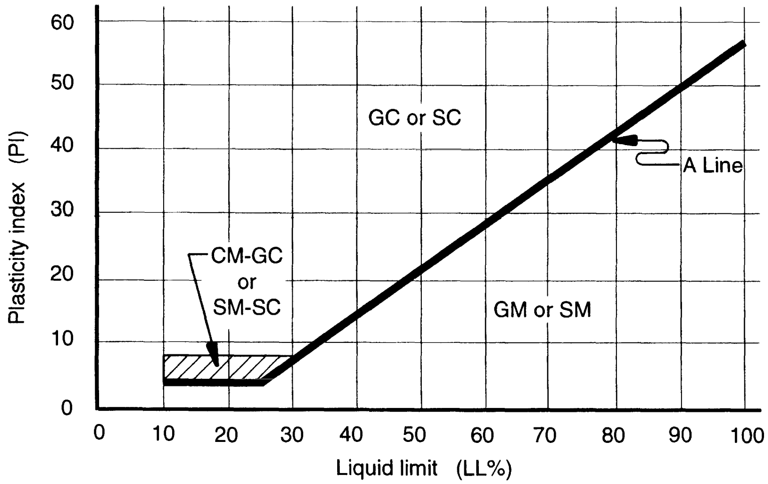


FIGURE 1-11. Plasticity chart, used in the identification of coarse grained soils.

ASTM Designation D-2487: Standard Test Method for Classification of Soils for Engineering Purposes

Although occasionally revised in the years since its adoption, ASTM D-2487 remains essentially the Unified System.

1-11. CLOSURE

It must be remembered that no system of classification is perfect nor will any system answer all questions or solve all problems. Each system must be used with care and only for the purpose intended. As a case in point, consider the textural system, as developed by the United States Department of Agriculture. According to this system, a soil identified as clay need only be a little more than 40% clay, with the remaining 60% made up of silt and sand in any number of proportions. Clearly, this system cannot be used alone as the basis for serious soil analysis.

The USCS, on the other hand, is much more technical in its system of classification. When an inspector calls in from the field and describes a particular soil as belonging to Group Symbol SW, the designer knows that the soil is identified as:

SW: Well-graded sands and gravelly sands, little or no fines

and he should have a relatively good understanding of the general character of the soil being described.

A preliminary judgment can also be made at this time as to which types of foundation may be best suited for this particular site. Final design, however, cannot be undertaken until more definitive information has been determined regarding the properties and load bearing characteristics of the soil.

Adequate, cost effective foundations can only be designed after the immediate and long-term response of the soil to vertical and lateral load has been determined. This information can only be obtained through a carefully thought out program of field and laboratory testing. The requirements for implementation of this program are described in Chapter 3.

1-12. SAMPLE PROBLEMS

Soils in the examples that follow will be identified in accordance with the Unified Soil Classification System.

Example 1-1

Required: To identify a given soil, based on the sieve analysis indicated in Table 1-3.

(a) An examination of the sample indicates no evidence of organic material.
(b) The weight of the soil contained on the Base represents the amount of soil passing all sieves, including the No. 200 sieve. Remember that soil passing through the No. 200 sieve is classified as fine grained soil. Soil retained on the No. 200 sieve or on any of the upper sieves is classified as coarse grained soil. In this particular example, then, 72 g are fine grained, and 1728 g are coarse grained. The coarse fraction is then computed as a percentage of the total dry weight:

$$\frac{1728}{1800} \times 100\% = 96\% < 50\%$$

Therefore, the soil is first classified as coarse grained.

TABLE 1-3. Example 1-1: Sieve Analysis

Sieve Size/No.	Weight Retained on Each Sieve (g)	Total Dry Weight Passing (%)
3″	0	100
3/4″	324	82
No. 4	648	46
No. 10	252	32
No. 40	216	20
No. 200	288	4
Base	72	0
	1800	

(c) The percentage of the coarse fraction retained on a No. 4 sieve is:

$$\frac{324 + 648}{1728} \times 100\% = 56\% > 50\%$$

Therefore, the soil is then classified as gravel.

(d) Because less than 5% of the sample passes a No. 200 sieve, a particle distribution curve must be drawn in order to determine the coefficients C_u and C_c. This will determine whether the gravel is well graded or poorly graded. The required particle distribution curve is shown in Figure 1-12.

(e) From the curve read: $D_{60} = 8.0$, $D_{30} = 1.7$ and $D_{10} = 0.13$

(f) From Formulas (1-1) and (1-2), compute:

$$C_u = \frac{8.0}{0.13} = 61.5 \qquad \text{and} \qquad C_c = \frac{1.7^2}{8.0 \times 0.13} = 2.78$$

These values are then compared to the following limiting values, which define a well graded gravel:

$$C_u > 4 \qquad \text{and} \qquad 3 > C_c > 1$$

Since these limits are satisfied, the soil should lastly be classified as:

GW: Well-graded gravels, gravel-sand mixtures, little or no fines.

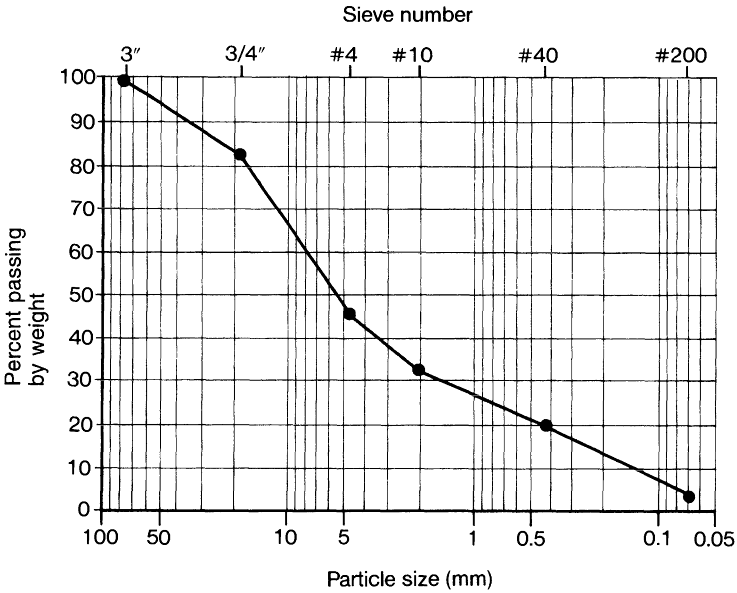


FIGURE 1-12. Example 1-1: Particle distribution curve.

TABLE 1-4. Example 1-2: Sieve Analysis

Sieve No.	Weight Retained on Each Sieve (g)	% of Total Dry Weight Passing
3″	0	100
3/4″	144	92
No. 4	540	62
No. 10	594	29
No. 40	126	22
No. 200	342	3
Base	54	0
	1800	

Example 1-2

Required: To identify a given soil based on the sieve analysis indicated in Table 1-4.

(a) An examination of the sample indicates no evidence of organic material.

(b) In this particular example, 54 g are fine grained, and 1746 g are coarse grained. The coarse fraction is then computed as a percentage of the total dry weight:

$$\frac{1746}{1800} \times 100\% = 97\% > 50\%$$

Therefore, the soil is first classified as coarse grained.

(c) The percentage of the coarse fraction retained on a No. 4 sieve is:

$$\frac{144 + 540}{1746} \times 100\% = 39\% < 50\%$$

Therefore, the soil is then classified as sand.

(d) Because less than 5% of the sample passes a No. 200 sieve, a particle distribution curve must be drawn in order to determine the coefficients C_u and C_c. This will determine whether the sand is well graded or poorly graded. The required particle distribution curve is shown in Figure 1-13.

(e) From the curve, read: $D_{60} = 4.4$, $D_{30} = 2.1$, and $D_{10} = 0.12$

(f) From Formulas (1-1) and (1-2), compute:

$$C_u = \frac{4.4}{0.12} = 36.7 \qquad \text{and} \qquad C_c = \frac{2.1^2}{4.4 \times 0.12} = 8.4$$

These values are then compared to the following limiting values, which define a well-graded sand:

$$C_u > 6 \qquad \text{and} \qquad 3 > C_c > 1$$

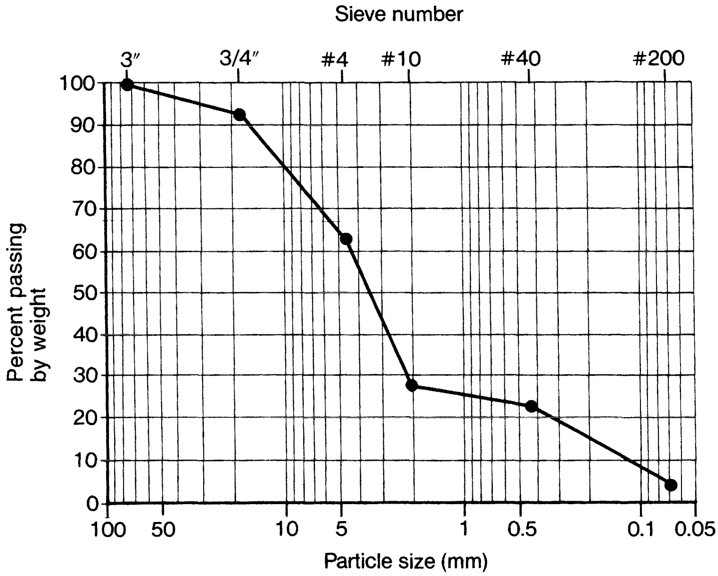


FIGURE 1-13. Example 1-2: Particle distribution curve.

Because these limits are not satisfied, the soil should lastly be classified as follows:

SP: Poorly graded sands or gravelly sands, little or no fines.

Example 1-3

Required: To identify a given soil based on the sieve analysis indicated in Table 1-5.

TABLE 1-5. Example 1-3: Sieve Analysis

Sieve Size/No.	Weight Retained on Each Sieve (g)	Total Dry Weight Passing (%)
3″	0	100
3/4″	412	77
No. 4	606	43
No. 10	155	35
No. 40	150	27
No. 200	202	15
Base	275	0
	1800	

(a) An examination of the sample indicates no evidence of organic material.
(b) In this particular sample, 275 g are fine grained, and 1525 g are coarse grained. The coarse fraction is then computed as a percentage of the total dry weight:

$$\frac{1525}{1800} \times 100\% = 85\% > 50\%$$

Therefore, the soil is first classified as coarse grained.
(c) The percentage of the coarse fraction retained on the No. 4 sieve is:

$$\frac{412 + 606}{1525} \times 100\% = 66\% < 50\%$$

Therefore, the soil is then classified as gravel.
(d) Because more than 12% of the sample passes a No. 200 sieve, a liquid limit and plastic limit evaluation must be made on the soil fraction passing a No. 40 sieve.
(e) A laboratory analysis provides the following information:

$$\begin{aligned} \text{Liquid limit } LL &= 44\% \\ \text{Plastic limit } PL &= 32\% \\ \text{Plasticity index } PI &= 12\% \end{aligned}$$

(f) The numerical values of liquid limit and plasticity index are plotted on the plasticity chart of Figure 1-11. Because the intersection point of these values falls below the A line, the soil should lastly be classified, according to Figure 1-8, as:

GM: Silty gravels, gravel–sand–silt mixture.

Example 1-4

Required: To identify a given soil based on the sieve analysis indicated in Table 1-6.

(a) An examination of the sample indicates no evidence of organic material.
(b) In this particular sample, 280 g are fine grained, and 1520 g are coarse grained. The coarse fraction is then computed as a percentage of the total dry weight:

$$\frac{1520}{1800} \times 100\% = 84\% > 50\%$$

Therefore, the soil is first classified as coarse grained.

TABLE 1-6. Example 1-4: Sieve Analysis

Sieve No.	Weight Retained on Each Sieve (g)	Total Dry Weight Passing (%)
3″	0	100
3/4″	198	89
4	450	64
10	468	38
40	162	29
200	242	16
Base	280	0
	1800	

(c) The percentage of the coarse fraction retained on the No. 4 sieve is:

$$\frac{198 + 450}{1520} \times 100\% = 43\% < 50\%$$

Therefore, the soil is then classified as sand.

(d) Because more than 12% of the sample passes a No. 200 sieve, a liquid limit and plastic limit evaluation must be made on the soil fraction passing a No. 40 sieve.

(e) A laboratory analysis provides the following information:

$$\text{Liquid limit } LL = 42\%$$
$$\text{Plastic limit } PL = 15\%$$
$$\text{Plasticity index } PI = 27\%$$

(f) The numerical values of liquid limit and plasticity index are plotted on the plasticity chart of Figure 1-11. Because the intersection point of these values falls above the A line, the soil should lastly be classified, according to Figure 1-8, as:

SC: Clayey sands, sand-clay mixtures.

Example 1-5

Required: To identify a given soil based on the sieve analysis indicated in Table 1-7.

(a) An examination of the sample indicates no evidence of organic material.

(b) In this particular sample, 1152 g are fine grained, and 648 g are coarse grained. The coarse fraction is then computed as a percentage of the total dry weight:

TABLE 1-7. Example 1-5: Sieve Analysis

Sieve Size/No.	Weight Retained on Each Sieve (g)	Total Dry Weight Passing (%)
3 inch	0	100
3/4 inch	0	100
No. 4	72	96
No. 10	126	89
No. 40	306	72
No. 200	144	64
Base	1152	0
	1800	

$$\frac{648}{1800} \times 100\% = 36\% < 50\%$$

Therefore, the soil is first classified as fine grained.

(c) A liquid limit and plastic limit evaluation must now be made on the soil fraction passing a No. 40 sieve. A laboratory analysis provides the following information:

$$\begin{aligned} \text{Liquid limit } LL &= 48\% \\ \text{Plastic limit } PL &= 33\% \\ \text{Plasticity index } PI &= 15\% \end{aligned}$$

(d) The numerical values of liquid limit and plasticity index are plotted on the plasticity chart of Figure 1-10. Because the intersection point of these values falls below the A line, the soil should lastly be classified as follows:

ML: Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.

Example 1-6

Required: To reevaluate the soil of Example 1-5, given a different laboratory analysis:

$$\begin{aligned} \text{Liquid limit } LL &= 55\% \\ \text{Plastic limit } PL &= 23\% \\ \text{Plasticity index } PI &= 32\% \end{aligned}$$

(d) The numerical values of liquid limit and plasticity index are plotted on the plasticity chart. Because the intersection point of these values falls above the A line, the soil should then be classified as follows:

CH: Inorganic clays of high plasticity, fat clays.

2

Physical Properties of Soils

2-1. GENERAL

The physical properties of a soil give insight as to the identification of the soil and the determination of its characteristics and load response. These properties can be determined by performing a laboratory analysis on undisturbed soil samples obtained during the test boring process, as described in Article 3-6. The laboratory analysis should be performed in accordance with the following ASTM Standard:

ASTM Designation D-854: Test Method for Specific Gravity of Soils

It should be noted that laboratory analyses are performed under controlled conditions with exacting materials and equipment. The results of such analyses may be considered to be accurate.

A field sample of undisturbed soil will contain three separate and distinct constituents: solids, water, and air. One of the important properties that must be determined in the laboratory analysis is that of the weight-volume relationship of these constituents. The makeup of this soil sample is illustrated as shown in Figure 2-1. In this figure and in subsequent work the following nomenclature is used:

W = weight Subscripts: a = air
V = volume w = water
v = voids (air + water)
s = solids

Seven Step Procedure for Weight–Volume Relationship of a Soil Sample

Step 1. Select the sample to be tested and determine its total volume V. The units of volume are usually cubic feet.

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

Step 2. Weigh the sample to determine its weight W, in pounds. Note that this weight includes both the weight of the water and the solid constituents.

Step 3. Determine the weight of the solid constituents W_s. Oven dry the sample at a constant temperature of 105 to 115°C. This will drive off all the free water from the sample. If the sample contains any particles of clay, this drying process will also remove any water molecularly bonded to those particles. The sample remaining after the drying process consists solely of solid constituents whose weight can now be determined.

Step 4. Determine the weight of water W_w, originally contained in the sample by subtracting the weight of the solids from the original weight of the sample:

$$W_w = W - W_s$$

Step 5. Now compute the volume of water V_w, corresponding to the weight of water found in Step 3. Remembering that density is the ratio of weight to volume, and that the density of water is 62.4 pcf, then:

$$V_w = \frac{W_w}{62.4}$$

Step 6. The volume of solids V_s may be determined by placing the solids from Step 3 into a container of known volume and filling the container with water whose volume is carefully measured. The difference between these two volumes represents the volume of the solids.

Step 7. The volume of air, if required, may be determined by subtracting the known volumes of water and solids from the initial total volume.

These weights and volumes should be recorded on a soil diagram similar to that depicted in Figure 2-1. They can then be used to determine important physical properties of the in situ soil from which the undisturbed sample was obtained.

2-2. UNIT WEIGHT

Unit weight, symbolized by (γ), is a term used to express the ratio of weight to the corresponding volume and is computed numerically in pounds per cubic foot.

$$\gamma = \text{unit weight} = \frac{W}{V} \text{ pcf}$$

Synonymous with the term unit weight is density. These terms are interchangeable. Unit weight is the term most frequently used in the laboratory, while density is the term with which architects, engineers and contractors are more familiar.

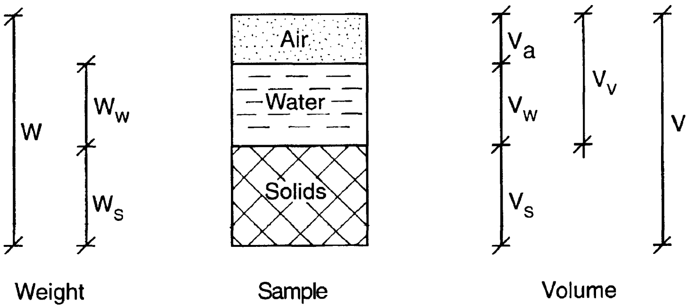


FIGURE 2-1. Weight–volume relationship of the constituents of a typical soil sample.

Soils generally contain air, water, and solids. By referring to Figure 2-1 the unit weight of any soil can be computed as follows:

$$\gamma = \frac{W_a + W_w + W_s}{V_a + V_w + V_s} = \frac{W_w + W_s}{V_v + V_s} = \frac{W_w + W_s}{V} \tag{2-1}$$

Variation in Unit Weight as a Function of Water Content

The unit weight of any volume of soil can be dramatically altered by varying the amount of water contained within it. Unit weights corresponding to three different water contents are of importance to the soils engineer. These conditions, as illustrated in Figure 2-2, are such that the volume of solids and the total volume of soil remain the same, but the water content in each sample is varied.

Sample 1. This sample is representative of the condition which occurs when all of the void normally filled with water has been replaced with air. The unit weight of this sample, therefore, is its least possible weight. This weight is called the dry weight and is symbolized by γ_{dry}.

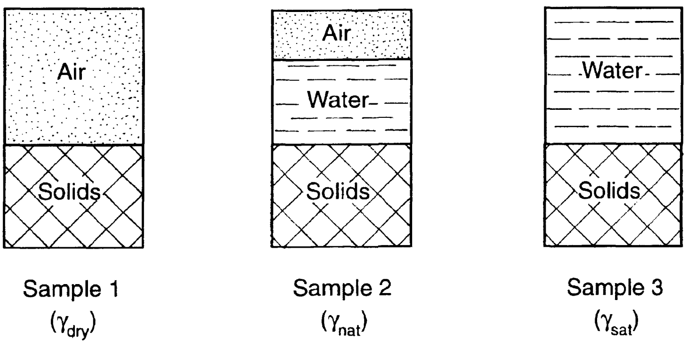


FIGURE 2-2. The effect of variation in water content on a given sample of soil.

Sample 2. This sample is representative of the in situ soil, which normally consists of air, water and solids. This weight is called the natural or in situ weight and is symbolized by γ_{nat} or simply by γ.

Sample 3. This sample represents the condition which occurs when all of the void normally filled with air has been replaced with water. The weight of this sample, therefore, is its greatest possible weight. This unit weight is called the saturated weight and is symbolized by γ_{sat}. This weight can be determined by adding water into the known volume until the voids are completely filled. The weight of this sample divided by the original volume will give the saturated unit weight.

Saturated and dry weights are important to the engineer because they provide him with the range within which the weight of the in situ soil can vary under the extreme conditions of flood and drought.

2-3. POROSITY

Porosity, symbolized by (n), is the term used to express the ratio of the volume of voids to the total volume of the mass:

$$n = \text{porosity} = \frac{V_v}{V} \tag{2-2}$$

Porosity is a property which depends on the physical characteristics of the soil particles, including their size, shape, uniformity, and arrangement within the mass. Because of the considerable influence that grain variation has on porosity, this property is not a true indication of whether a sandy soil is loose or dense or whether a clayey soil is soft or hard. These properties are discussed in detail in other sections of the text. Porosity is, however, an important property indicator of the permeability of the soil.

The permeability of various soils is given in Article 10-13. The importance of this property in selecting a soil based on drainage characteristics is discussed in Article 10-14.

2-4. VOID RATIO

Void ratio, symbolized by (e), is the term used to express the ratio of the volume of voids to the volume of solids:

$$e = \text{void ratio} = \frac{V_v}{V_s} \tag{2-3}$$

Void ratio can also be expressed in terms of porosity, by substituting $V - V_v$ for V_s and dividing all of the terms by V. This results in:

$$e = \text{void ratio} = \frac{n}{1 - n} \tag{2-4}$$

2-5. WATER CONTENT

Water content, symbolized by (w), is a term used to express, in percentage, the ratio of the weight of water to the weight of solids.

$$w\% = \text{water content} = \frac{W_w}{W_s} \times 100\% \tag{2-5}$$

Synonymous with the term water content is moisture content. These terms are interchangeable. Water content is the term most frequently used in the laboratory, while moisture content is the term with which architects, engineers and contractors are more familiar.

Closely related to water content is the degree of saturation (S), which expresses in percent the volume of water to the volume of voids:

$$S\% = \text{degree of saturation} = \frac{V_w}{V_v} \times 100\% \tag{2-6}$$

2-6. SPECIFIC GRAVITY

Specific gravity, symbolized by the letter (G), is the term used to denote the ratio of the weight of a substance to the weight of an equal volume of water:

$$G = \text{specific gravity} = \frac{W}{62.4\ \text{V}}$$

The specific gravity of the solid constituents of a soil is:

$$G_s = \frac{W_s}{62.4\ V_s} \tag{2-7}$$

The specific gravity of the solid constituents of a soil has been found to vary between wide limits. The precise value can only be determined by laboratory analysis. In the absence of such an analysis, the following values can be used as reasonable approximations:

TABLE 2-1. Physical Properties of Typical Soils in Natural State [Ref. 16]

Soil Description	n	e	$w\%$	γ_{dry}	γ_{sat}
Uniform sand, loose	0.46	0.85	32	90	118
Uniform sand, dense	0.34	0.51	19	109	130
Mixed-grained sand, loose	0.40	0.67	25	99	124
Mixed-grained sand, dense	0.30	0.43	16	116	135
Glacial till, mixed-grained	0.20	0.25	9	132	145
Soft glacial clay	0.55	1.20	45	76	110
Stiff glacial clay	0.37	0.60	22	106	129
Soft, slightly organic clay	0.66	1.90	70	58	98
Soft, very organic clay	0.75	3.00	110	43	89
Soft bentonite	0.84	5.20	194	27	80

G_s for sands and gravels: 2.65 to 2.68
G_s for silts and clays: 2.58 to 2.75

2-7. REPRESENTATIVE VALUES OF PHYSICAL PROPERTIES

Representation values of porosity, void ratio, water content, and unit weight for several different kinds of soil are listed in Table 2-1.

2-8. RELATIVE DENSITY

Relative density, symbolized by (D_r), is a property belonging exclusively to coarse grained soils having no fines or having fines of insufficient quantity to impart any measurable cohesion to the soil. One of the most important properties of a coarse grained soil is its degree of compaction. This property exerts considerable influence on the ultimate bearing capacity and settlement characteristics of a natural deposit of this kind of soil.

The degree of compaction is a representation of the unit weight of the soil. It is the nature of soils that the unit weight of the solid constituents is greater than that of water. The unit weight of the soil, therefore, will increase or decrease depending upon the amount of solids contained within a given volume. Tightly packed soils will contain more solids and will have less voids than a loosely packed soil, given the same volume. The relative amount of solids contained within the same volume for loosely and densely packed soils is illustrated in Figure 2-3. Note that the term voids as used in Figure 2-3 may include both air and water.

Sample 1. This sample is representative of a very loosely packed soil having maximum voids. The void ratio is e_{max}. Note that maximum voids corresponds to minimum possible density.

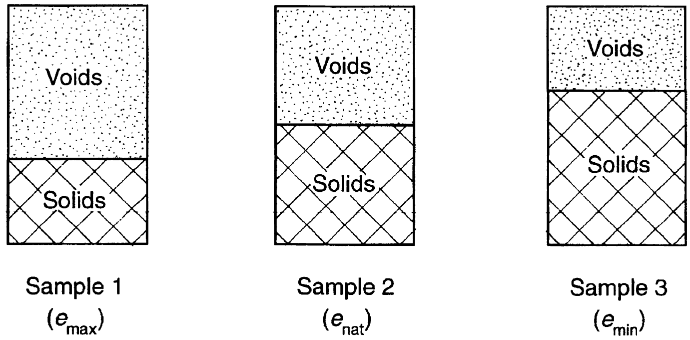


FIGURE 2-3. The effect of variation in void ratio on a given sample of coarse grained soil.

Sample 2. This sample is representative of the in situ soil. The void ratio is e_{nat}.
Sample 3. This sample is representative of a very tightly packed soil having minimum voids. The void ratio is e_{min}. Note that minimum voids corresponds to maximum possible density.

Relative density is a term used to numerically compare the density of an in-place natural or compacted soil with the densities representative of the same soil in the extreme states of looseness and denseness. Relative density can be expressed in terms of void ratio, as indicated:

$$\text{Relative density} = D_r = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}} \times 100\% \qquad (2\text{-}8)$$

Where
e_{max} is the void ratio of the sample in its loosest state
e_{nat} is the void ratio of the sample in its natural state
e_{min} is the void ratio of the sample in its densest state

By examining the upper and lower limits of Formula (2-8), it can be seen that the relative density numerically approaches 0% for a very loose soil and 100% for a very dense soil. It can also be seen that a soil with a relative density of 50% has a void ratio midway between the void ratios of the same soil in the extreme state of looseness and denseness.

In order to compute the relative density of an in-place soil by the use of Formula (2-8), it is necessary to determine the three previously described void ratios. These ratios shall be based on measurements of weight and volume taken in the laboratory on representative samples of in-place soil obtained from the field. Such measurements shall be in accordance with Steps 1 through 6 of Article 2-1. Void ratios may then be computed in accordance with Article 2-4. Sample 1, representing the loosest possible state, is prepared by allowing the soil to

TABLE 2-2. Representative Values of Relative Density for Coarse Grained Soils [Ref. 13]

Description	$D_r\%$	Density (pcf)
Loose	<35	<90
Medium	35–65	90–110
Dense	65–85	110–130
Very dense	>85	>130

gently free fall into a container of known volume until such time that the container is filled. Sample 3, representing the densest possible state, is prepared by packing the soil into the container until the container will accept no more soil. Sample 2, representing the soil in its natural state, is evaluated by working on an undisturbed sample.

The numerical accuracy of the three required void ratios depends on the care with which the tests are performed. In the test for the loosest state, the height of free fall must be sufficient to allow for the free separation of the particles, but it must not be so high that the particles compact while filling the container. In the test for the densest state care must be taken to compact the soil in layers and to thoroughly compact each layer.

Representative values of relative densities for various degrees of soil compaction are given in Table 2-2.

Relative density is of primary importance in problems involving compaction of coarse grained soils. That situation is discussed in detail in Article 12-6. In that article there is a procedure whereby relative density can be computed using unit weights (density), instead of void ratios.

2-9. UNCONFINED COMPRESSION STRENGTH

Unconfined compression strength, symbolized by (q_u), is a property belonging exclusively to clay and to mixed grained soils of which clay is the predominant fraction. This property relates to clay as the ultimate compression strength (f'_c) relates to concrete. As in the case of concrete, this property can be estimated, but its true value can be determined only by laboratory analysis. Unconfined compression strength is of primary importance in the design of spread footings because it is the determining factor in establishing the ultimate bearing strength of the soil.

The required laboratory analysis is performed on an undisturbed sample of clay whose height is usually 1½ to 2 times its diameter. The sample is subjected to a compressive force of increasing intensity until failure occurs. Failure in this test is defined as whichever of the following is the first to occur:

a. literal breaking of the sample
b. a shortening of height equaling 15% axial strain

This procedure must be performed quickly otherwise moisture may permeate out of the sample thereby invalidating the test. The unconfined compression strength is found by dividing the load at failure by its cross sectional area. All procedures in this analysis must be performed in accordance with the following ASTM Standard:

> ASTM Designation D-2166: Unconfined Compression Strength of Cohesive Soil

The typical apparatus used in this test procedure is shown in Figure 2-4.

It should be of interest to note that literal failure of a sample during an unconfined compression test is actually caused by shear rather than by direct

FIGURE 2-4. An unconfined compression test in progress.

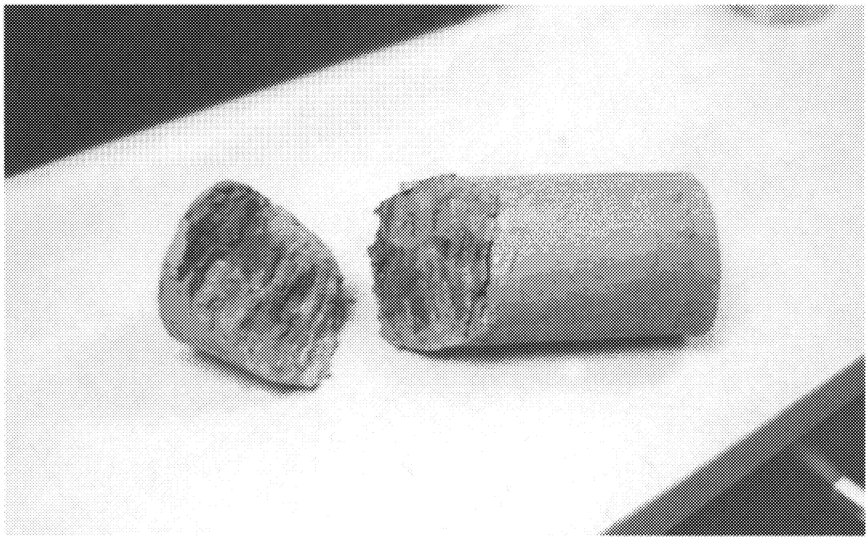


FIGURE 2-5. The result of the unconfined compression test of Figure 2-4, showing mode of failure.

compression. This can be anticipated from Figure 2-4, where a failure on a diagonal plane appears imminent. The same sample after failure, as shown in Figure 2-5, confirms that failure occurred on a diagonal plane. This is representative of a failure due to shear.

2-10. CONSISTENCY

Consistency is a term applied to cohesive soils to describe the degree with which a particular soil will resist deformation. The settlement of a building is directly related to the deformation of the soil upon which it bears. If the soil did not deform, there would be no settlement. The consistency of a soil, therefore, is a measure of the load carrying ability of that soil. Comparative terms such as soft, medium, stiff, and hard are in common usage, but it must be remembered that these terms may have different meanings in different parts of the country.

It has been determined that for soils rich in clay, there is a definable correlation between consistency and unconfined compression strength. This correlation is given in Table 2-3.

2-11. SENSITIVITY

Sensitivity (S_t) relates to clay or to soils whose characteristics are those predominantly of clay. Sensitivity is the term used to measure the relative loss of strength

TABLE 2-3. Quantitative Expressions for Consistency of Clays [Ref. 16]

Consistency	Field Examination	q_u (tsf)
Very soft	Easily penetrated several inches by fist	<0.25
Soft	Easily penetrated several inches by thumb	0.25–0.50
Medium	Can be penetrated several inches by thumb with moderate effort	0.50–1.00
Stiff	Readily indented by thumb but penetrated only with great effort	1.00–2.00
Very stiff	Readily indented by thumbnail	2.00–4.00
Hard	Indented with difficulty by thumbnail	>4.00

experienced by a soil when subjected to a dynamic disturbance. Such a disturbance can be of natural origin, as would occur during an earthquake, or can be man-made, as would occur during the construction of a building.

Sensitivity is measured in the laboratory by running unconfined compression tests on undisturbed samples and also on samples that have been remolded. The purpose of remolding is to produce the same effect on the laboratory sample as would be produced by a dynamic disturbance on an in situ soil mass. Remolding is a term used to indicate the physical manipulation of a previously undisturbed sample of clay by kneading and working it in the hands. Such a clay is referred to as a remolded clay. When a sample of clay is remolded at a constant moisture content it will become softer and easier to work.

Sensitivity may be expressed numerically as follows:

$$\text{Sensitivity} = S_t = \frac{q_u \text{ of undisturbed sample}}{q_u \text{ of remolded sample}}$$

Sensitivity tests performed in the laboratory evaluate the relative loss in strength of a sample after it has been remolded. A sensitivity factor of 1 indicates that the strength of the remolded sample is equal to that of the undisturbed sample. Therefore, there is no loss in strength in that particular soil. Sensitivity factors of 2 to 4 are considered normal. Soils with higher sensitivity factors are at progressively higher risk in the event of a dynamic disturbance. Highly sensitive soils are referred to as quick. Such soils must be treated with care because any kind of dynamic disturbance could cause the soil to transform, at least temporarily, into a viscous liquid.

A loss in compression strength is always accompanied by a corresponding loss in shear strength. For this reason highly sensitive soils are very prone to landslide. This phenomenon can be particularly dangerous when excavating for a building project within or adjacent to highly sensitive soils. Quick soils are particularly susceptible to landslide due to a tendency on the part of the soil to temporarily liquefy when subjected to sudden shock. There are recorded instances of very quick soils experiencing treacherous landslides on very gentle slopes. It is also noted that landslides occur quickly with little warning as to impending danger.

In clays having high sensitivity, the particles are usually arranged in the form of loose but relatively stable structures called flocs. The structure of the floc is such that its volume is very large compared to that of the solid constituents. Clays having a flocculent structure, therefore, can experience large decreases in volume when subjected to shock or vibration. Soils consisting of highly sensitive clays should never be used to provide either vertical or lateral support to any part of a building structure because of the very real possibility of excessive building settlement.

Sensitivity is an important consideration during the construction of a building whenever the soil at the site or adjacent to the site is going to be subjected to any kind of shock or vibration. This could occur in any number of situations, including:

1. The operation of heavy machinery during excavation
2. The use of dynamite to dislodge or breakup rock or large stones
3. The use of percussion drills when installing tie backs
4. The driving of timber or steel piles

Safety of materials and personnel during all construction processes is the responsibility of the contractor. It is the responsibility of the architect and engineer, however, to make all known site and subgrade information available to the contractor. The contractor may, at his own expense, have additional test borings and soil tests performed under his direction and for his own purpose.

2-12. ANGLE OF INTERNAL FRICTION

The angle of internal friction, symbolized by (ϕ), is one of the most important physical properties of a coarse grained material. It is used extensively in the theories relating to allowable soil bearing pressure, as developed in Chapter 5, and to lateral earth pressure, as developed in Chapter 9.

This angle is a measurement of the ability of the coarse grained fraction of a soil to resist shear through intergranular friction. The magnitude of this angle depends on several factors, including the size, shape, and distribution of the grains, as well as the moisture content and the degree of compaction. Laboratory tests have shown that the angle of internal friction is larger for soils whose grains are angular rather than rounded and is larger for soils having a wide range of grain size rather than a uniformity of size. Mixed grained soils also exhibit a measurable angle of internal friction, the magnitude of which depends upon the ratio of coarse to fine grained fractions. The magnitude decreases as the percentage of fines increases. A fine grained soil does not possess this property at all.

The magnitude of the angle of internal friction should be determined by laboratory analysis performed on undisturbed samples of the actual soil in question. These samples must truly reflect the same moisture content and degree of compaction that will exist during service, otherwise such tests will be valueless.

TABLE 2-4. Angle of Internal Friction for Representative Coarse Grained Soils [Ref. 13]

Soil Type	Angle Range
Sand and gravel mixture	33–36°
Well graded sand	32–35°
Fine to medium sand	29–32°
Silty sand	27–32°
Nonplastic silt	26–30°

In the absence of a laboratory analysis, the angle of internal friction can be approximated from the general information given in Table 2-4.

2-13. COHESION

Cohesion symbolized by (c), is a property by which particles of clay exhibit a measurable amount of stickiness. It is this property by which these soils develop resistance to shear. Soils exhibiting this property are referred to as cohesive soils. Mixed grained soils having appreciable fines may also exhibit this property although to a lesser degree.

The resistance to shear developed by cohesion is independent of any normal force that may exist on the plane of rupture. Cohesive soils, therefore, develop resistance to shear in a completely different way than do sands and gravels whose resistance depends solely on the physical interlocking of the individual soil grains.

Cohesion is a molecular phenomenon unique to clay, in which each molecule carries a tiny surface charge of electricity which attracts and holds the molecules together. Cohesion can be thought of being much like a glue which produces resistance to shear by bonding the surfaces together. Numerous tests have determined that the numerical value of cohesion can be taken as one-half of the unconfined compression strength q_u of the soil.

2-14. SAMPLE PROBLEMS

Example 2-1

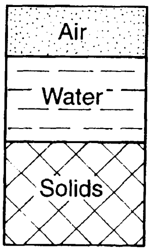
Required: To determine the physical properties of a particular sample of a coarse grained soil. This is an undisturbed sample and is representative of an in situ soil deposit. Determine the volume/weight relationship of the soil using the procedure outlined in Article 2-1.

Step 1. The volume V of the container is 1 CF.
Step 2. The weight W of the soil sample is 117.6 lb.
Step 3. The weight W_s of the solids of the oven dry sample is 105.0 lb.
Step 4. The weight W_w of water is 117.6 − 105.0 = 12.6 lb.
Step 5. The volume V_w of water is 12.6/62.4 = 0.202 CF.
Step 6. Using the same container as before, the volume of water required to fill the container was measured as 0.370 CF. Then compute:

$$\text{Volume of solids } V_s = 1.000 - 0.370 = 0.630 \text{ CF}$$

Step 7. Volume of air $V_a = 1.000 - 0.202 - 0.630 = 0.168$ CF

These results are indicated in Figure 2-6.



Volume (cf)		Weight (#)
0.168	Air	0.0
0.202	Water	12.6
0.630	Solids	105.0
1.000	Σ	117.6

FIGURE 2-6. Example 2-1—Soil sample.

The physical properties of this soil are now computed:

$$\gamma = \frac{117.6}{1.000} = 117.6 \text{ pcf} \tag{2-1}$$

$$n = \frac{0.370}{1.000} = 0.370 \tag{2-2}$$

$$e = \frac{0.370}{0.630} = 0.587 \tag{2-3}$$

$$w = \frac{12.6}{105.0} \times 100\% = 12.0\% \tag{2-5}$$

$$S = \frac{0.202}{0.370} \times 100\% = 54.6\% \tag{2-6}$$

$$G_s = \frac{105.0}{62.4 \times 0.630} = 2.67 \tag{2-7}$$

Example 2-2

Required: To determine the saturated and dry unit weights of the soil previously analysed in Example 2-1. Referring to Figure 2-2:

Sample 1. In Step 3, all of the water was removed from the sample leaving only the solids. The dry unit weight of this sample is, therefore:

$$\gamma_{dry} = 105.0 \text{ pcf}$$

Sample 2. In Step 2 the unit weight of the in situ soil was found to be:

$$\gamma_{nat} = 117.6 \text{ pcf}$$

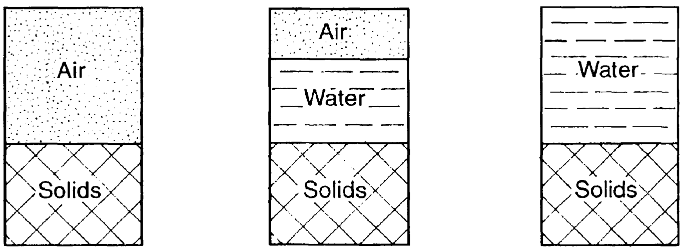
Sample 3. In Step 6 the volume of voids of the oven dry sample was found to be 0.370 CF. The soil is saturated when this entire volume is filled with water. The weight of this water is:

$$0.370 \times 62.4 = 23.1\#$$

The saturated weight of the sample is the sum of the solids and the water:

$$\gamma_{sat} = 105.0 + 23.1 = 128.1 \text{ pcf}$$

The results of these calculations are shown in Figure 2-7.



	Sample 1	Sample 2	Sample 3
Air	0.0	0.0	0.0
Water	0.0	12.6	23.1
Solids	105.0	105.0	105.0
Σ	105.0	117.6	128.1

FIGURE 2-7. Example 2-2—Soil sample.

Example 2-3

Required: To determine the relative density D_r of the soil previously analyzed in Example 2-1.

Two new samples of soil must be prepared. The container with the first sample must be very loosely packed. The container with the second sample must be very tightly packed. Then refer to Figure 2-3 and perform the indicated measurements and calculations on each sample:

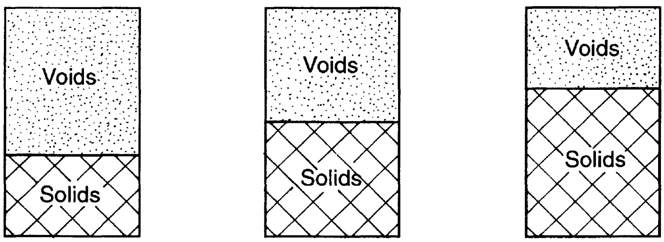
	Loosely packed	Tightly packed	
Step 1. V	1.000 CF	1.000 CF	
Step 3. W_s	87.6 lb	113.3 lb	(weighed)
Step 6. V_v	0.474 CF	0.320 CF	(measured)
V_s	0.526 CF	0.680 CF	$(V - V_v)$
e	0.901 max	0.471 min	(V_v/V_s)

The results of this analysis are shown in Figure 2-8. The relative density of the in situ soil is now computed using Formula (2-8):

$$\text{Relative density } D_r = \frac{0.901 - 0.587}{0.901 - 0.471} \times 100\% = 73.0\%$$

This soil can be classified as mid-range dense soil according to the terminology of Table 2-2.

Relative density can also be computed by using unit weights instead of void ratios. This alternate procedure is described in Article 12-6. In order to use this alternate procedure, however, the weight W of the soil sample as determined in Step 2 will be required.



	Sample 1	Sample 2	Sample 3
Voids	0.474	0.370	0.320
Solids	0.526	0.630	0.680
Σ	1.000	1.000	1.000
e	0.901	0.587	0.471

FIGURE 2-8. Example 2-3—Soil sample.

Example 2-4

Required: To determine the physical properties of a particular sample of a saturated, fine grained soil. This is an undisturbed sample, and is representative of an in situ soil deposit. Determine the volume/weight relationship of this soil using the procedure outlined in Article 2-1.

Note. This is a saturated soil. All voids are filled with water.

Step 1. The volume V of the container is 1 CF.
Step 2. The weight W of the soil sample is 113.4 lb.
Step 3. The weight W_s of the solids of the oven dry sample is 82.7 lb.
Step 4. The weight W_w of water is 113.4 − 82.7 = 30.7 lb.
Step 5. The volume V_w of water is 30.7/62.4 = 0.492 CF.
Step 6. Water measurement is not required because soil is saturated.

$$\text{Volume of solids } V_s = 1.000 - 0.492 = 0.508 \text{ CF}$$

These results are indicated in Figure 2-9.
The physical properties of this soil are now computed:

$$\gamma = \frac{113.4}{1.000} = 113.4 \text{ pcf} \quad (2\text{-}1)$$

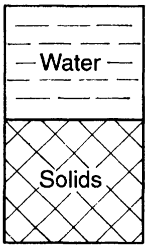
$$n = \frac{0.492}{1.000} = 0.492 \quad (2\text{-}2)$$

$$e = \frac{0.492}{0.508} = 0.969 \quad (2\text{-}3)$$

$$w = \frac{30.7}{82.7} \times 100\% = 37.1\% \quad (2\text{-}5)$$

$$S = 100\% \text{ (saturated soil)} \quad (2\text{-}6)$$

$$G_s = \frac{82.7}{62.4 \times 0.508} = 2.61 \quad (2\text{-}7)$$



Volume (cf)		Weight (#)
0.492	Water	30.7
0.508	Solids	82.7
1.000	Σ	113.4

FIGURE 2-9. Example 2-4—Soil sample.

Example 2-5

Required: To determine the density, void ratio and degree of saturation of a particular in situ soil identified as a lean, stiff clay. The following data were taken from a testing laboratory report.

$$\gamma_{\text{dry}} = 106.4 \text{ pcf}, \qquad G_s = 2.68, \qquad w = 20.1\%$$

The soil density is determined as follows:

Noting that $\gamma_{\text{dry}} = W_s$:

$$V_s = \frac{106.4}{62.4 \times 2.68} = 0.636 \text{ CF} \tag{2-7}$$

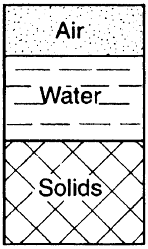
$$V_v = 1.000 - 0.636 = 0.364 \text{ CF}$$

$$W_w = 0.201 \times 106.4 = 21.4 \text{ lb} \tag{2-5}$$

$$V_w = \frac{21.4}{62.4} = 0.343 \text{ CF}$$

$$V_a = 0.364 - 0.343 = 0.021 \text{ CF}$$

These results are indicated in Figure 2-10.



Volume (cf)		Weight (#)
0.021	Air	0.0
0.343	Water	21.4
0.636	Solids	106.4
1.000	Σ	127.8

FIGURE 2-10. Example 2-5—Soil sample.

The density of the in situ soil is therefore 127.8 pcf.

The void ratio and degree of saturation are computed as follows:

$$e = \frac{0.364}{0.636} = 0.572 \tag{2-3}$$

$$S = \frac{0.343}{0.364} \times 100\% = 94.2\% \tag{2-6}$$

3

Subsurface Soil Exploration

3-1. PRELIMINARY INVESTIGATION OF SITE

General Considerations

At the earliest possible time during the development of a building project, the architect, along with the engineer, should make a preliminary investigation of the site. The purpose of this investigation is to acquaint both the architect and engineer with conditions peculiar to that site. The investigation will give the architect information as to site access, topography, and the existence of physical barriers such as sharp drop-offs, lakes, or streams of running or stagnant water.

The architect must also determine whether public gas, electric, water, and sewage disposal are available, and in what quantities. If public water is not available, tests must be conducted to determine the depth to ground water and whether the quantity and rate with which that water can be recovered is sufficient for the needs of the proposed project. If public sewage is not available, the ability of the site to accept on-site sewage disposal must be determined. When public water and/or sewage is not available, a study must be made to determine the environmental effect of drawing the large quantities of water required from the ground water pool and of disposing similar quantities of sewage into the ground through on-site disposal systems. These studies must be conducted in conjunction with the appropriate governmental agencies.

On occasion the developer will ask for a preliminary investigation by the architect before formally acquiring the land. There have been instances in which the findings of such a preliminary investigation indicate that the land is unsuitable for the purpose intended.

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

Prior to going to the site, the architect should seek out and examine in detail all known documentation regarding the site. This may include, but is not necessarily limited to, the following general sources:

1. *Maps.* General area and large scale detail maps.
2. *Aerial Photographs.* These may be available from local utility companies or from governmental sources. In special cases it may be considered desirable to recommend that the owner engage the services of an aerial photographer.
3. *Satellite Imagery.* This specialized information is readily available from independent sources and from governmental agencies such as the Department of Agriculture.
4. *Geological Survey Maps.* Produced by and available from the United States Government, these maps are an excellent source of generalized information.
5. *Soil Conservation Maps.* Produced by and available from the United States Department of Agriculture, these maps generally include information as to the classifications of soils and their suitability for various uses.
6. *Adjacent Site Investigations.* Inquiries should be made to see if any similar investigations have been made on sites near or adjacent to the site in question.
7. *Utilities.* Local utility companies such as gas, electric, water, telephone, etc., should be contacted regarding the existence of any underground utilities on or near the site.

The architect should visit the site with camera in hand. He should walk the entire site and should record all significant features. Particular attention should be given to site access for the future work of surveyors, test borings, and, ultimately, construction.

Some architects own a transit and use this at the site to obtain preliminary information regarding general site topography. When a site has a very gentle slope it can appear to be level. A transit will determine its true characteristics. This investigation is in no way meant to replace the formal survey that is required for all projects and which must be made by professional surveyors. If this survey has already been made, the architect, of course, need not duplicate the work.

Portable Sampling Equipment

There are numerous times when the architect wants to make a very preliminary exploration of the underlying soil. Several lightweight, portable drilling tools are available to the architect for that purpose. These tools are easily operated by one man and can conveniently be transported and stored in the trunk of a car.

The Iwan type auger, as illustrated in Figure 3-1, is sometimes referred to as a post hole digger. It is a very effective hand operated drilling tool used in the recovery of samples of earth. Its use, however, is limited to soils having sufficient

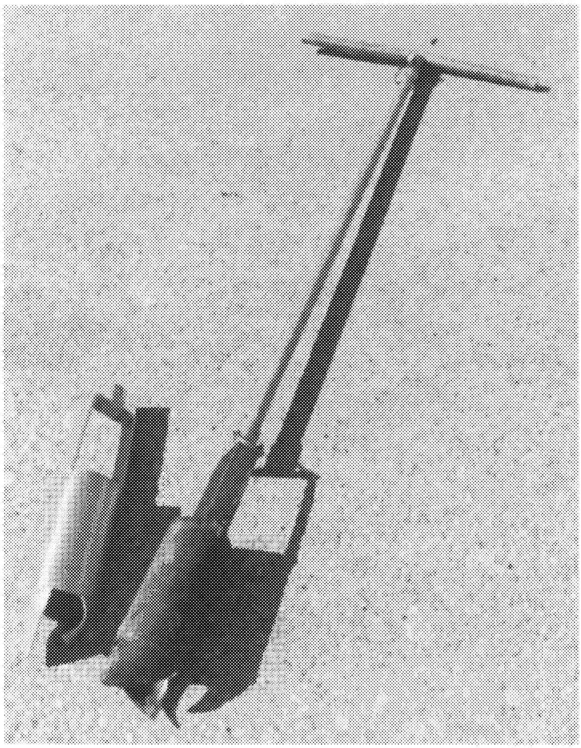


FIGURE 3-1. Post hole auger, Iwan type, for procuring samples by hand.

cohesion that the side walls of the drill hole will not collapse. This auger is usually equipped with a cutting blade having a 3-inch diameter. With favorable soil conditions, samples can be recovered from depths of up to twenty-five feet. The Iwan type auger is operated by pushing the cutting blade into the ground while turning it as one would turn a screw. The sample is recovered by lifting the auger out of the ground. Samples at lower depths can be recovered by adding extensions to the auger shaft. The samples obtained by using the Iwan auger will be somewhat chewed up but are usually adequate for generalized identification.

The ship auger, as illustrated in Figure 3-2, is similar to the Iwan type auger in ease of use and portability. It differs, however, in construction and use. The drilling end of the ship auger is an open spiral screw having a usual diameter of 2 inches. After drilling to the desired depth, the auger is lifted out of the ground, and any material caught within the spiral will be brought to the surface for examination. Although this auger can be drilled through granular material it can only recover samples of cohesive soil.

FIGURE 3-2. Ship auger, for procuring samples by hand. [Ref. 1]

Soil Examination by Test Pit

The architect will frequently request that the owner authorize the use of test pits for preliminary site investigation. Test pits are open holes dug at the site for the purpose of examining the soil in situ. These pits are preferably dug with a back-hoe trencher, as illustrated in Figure 3-3.

The advantage of using a back-hoe, rather than digging a test pit by hand or by clam shovel, is that this equipment is very quick, mobile, and versatile. It will excavate a trench anywhere from 10 to 18 feet in depth, depending on the equipment, and can extend the trench for any desired length. These open cuts can be very helpful in giving a general indication of the different strata of soil occurring within that depth.

When the excavation is made in granular material the side walls, of course, will not stand unless shored. In this case the soil must be examined as it is

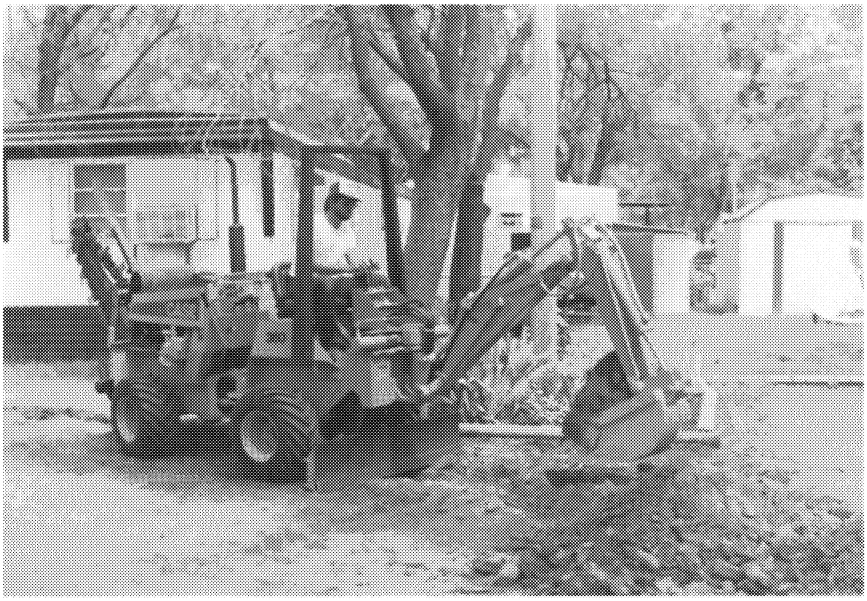


FIGURE 3-3. A backhoe in the process of cutting a trench for visual examination of a soil profile. [Ref. 7]

brought to the surface by the back-hoe. Although this soil will be disturbed and somewhat mixed, it will still give the engineer insight as to the character of the soil and of its variation with depth.

Excavation in cohesive soil affords the engineer with the possibility of a hands-on examination of the soil in an undisturbed condition, as indicated in Figure 3-4. Once it has been established that the side walls will not collapse, the engineer can climb down into the trench and more closely examine the exposed soil in its natural state. Samples of the different materials encountered can be taken at this time, if desired, and kept for future reference.

A word of caution: the architect or engineer in charge of the work should make absolutely certain that all test pits are backfilled and reasonably compacted before the work is ended for each day. He should see to this himself and not rely on the intentions of others. No excavation should ever be permitted to remain open overnight or unattended for any time whatsoever.

3-2. FIELD SURVEY

If the owner recently purchased the property, a field survey will have been made immediately before legal transfer. Such a survey would have been required by

FIGURE 3-4. A test pit in cohesive soil ready for examination.

the title company. If there is no record of such a survey or if an existing one is considered to be out of date, then a new survey should be authorized.

If a new survey is required, the architect or his representative will interview several licensed land surveyors who work in the general area of the site. After a surveyor has been selected and placed under contract by the owner, the actual field work will begin. The work should generally include the following:

1. Research and examination of available drawings and other documents pertinent to the work
2. A physical survey of the property
3. Preparation of a scaled and detailed drawing of the property, which will include:
   a. A mathematical description of the boundaries of the property, called a metes and bounds

b. A topography of the site showing contours and elevations relating to a known and physically identifiable data
c. A layout of all existing barriers and obstructions, including those that are of natural origin and those that are man-made
d. A layout of all known underground or overhead utilities

3-3. PRELIMINARY IDENTIFICATION OF SOILS

Common Usage of Soil Terms

Soils are technically classified as coarse grained or fine grained, depending on particle size. A coarse grained soil is essentially granular and this classification is frequently referred to as a granular soil. Because the dominant grain size in a granular soil is more likely to be sand, rather than gravel a soil having granular characteristics, may be referred to as sand. The dominant characteristic of a fine grained soil is cohesion. Fine grained soils, therefore, are frequently referred to as cohesive soils. Because cohesion is a singular property of clay any soil exhibiting a marked cohesion may loosely identified as a clay.

Visual Identification

The samples of earth recovered by the architect and engineer during their initial exploration of the site can be used to identify the general composition of the soil in terms of gravel, sand, silt and clay.

Sand and gravel are easy to visually identify because the individual particles can readily be seen with the naked eye or with a small magnifying glass. Individual particles of silt and clay, however, are much too small to be seen in this way and, therefore, require other means of identification.

Soils that are predominantly silt or clay exhibit more of the individual properties associated with the predominant material. For soils that are rich in silt or clay, the following tests can be used to identify the soil from its individual properties. These tests are relatively easy to perform and can be made in the field, while the test pits are still open.

Touch Test

Silts and clays have a slightly different feel to the touch and, although not conclusive, this test may be helpful in identifying the more prevalent material. Silt is somewhat gritty to the touch and dusts off easily when dried on the hands. Clay, when moist, has a tendency to be sticky and when dried on the hands does not brush off easily.

Dry Strength Test

This test is used to distinguish silts from clays by comparing the relative strengths of dry samples.

A sample of soil is first allowed to dry in the air and is then broken into small fragments that are pressed between the thumb and forefinger. Fragments that are predominantly silt will break easily, but fragments of clay can be broken only with much more effort.

Shaking Test

Silts are more permeable than clays. The shaking test is used to identify the soil by means of its permeability characteristics.

A sample of soil is mixed with water into a very soft consistency in the palm of the hand. As the back of the hand is lightly tapped from underneath, water from within the soil will permeate to the surface. Then, when the sample is manipulated with the fingers and remolded, the water will permeate back into the soil. If the soil is primarily silt, the movement of water in each test will be much quicker than if the soil is primarily clay. In the case of pure clay, there will be little or no water activity.

Thread Test

The thread test is a test of plasticity, which is the property whereby a soil can withstand large deformations without breaking. When a sample of moist soil contains significant amounts of clay, it can be rolled out into a long, thin thread, and when suspended between the fingers it will support its own weight. Silt, on the other hand, can seldom be rolled out into a thread without severe cracking, nor will it exhibit any measurable tensile strength.

Dispersion Test

This is a test of the relative time during which silt and clay particles will remain in suspension. A sample of soil is thoroughly mixed with water and allowed to settle. Silt, which consists of coarser microscopic particles, will settle out first, while clay particles will remain in suspension for a much longer time.

Representative Test Results

The results which can generally be expected from performing the foregoing tests on different kinds of soil are given in Table 3-1. These results can be used as general guidelines in identifying soils that are rich either in silt or in clay.

TABLE 3-1. The Response of Soils to Simple Field Tests [Ref. 16]

Sample	Dry Strength	Shaking	Thread	Dispersion
Sandy silt	None	Rapid	None	sec/min
Silt	Low	Rapid	Weak	min
Clayey silt	Low/med	Rapid/slow	Medium	min/hr
Sandy clay	Low/high	Slow/none	Medium	sec/hr
Silty clay	Med/high	Slow/none	Medium	min/hr
Clay	High	None	Tough	hr/days
Organic silt[a]	Low/med	Slow	Weak	min/hr
Organic clay[a]	Med/high	None	Tough	hr/days

[a] Silts and clays which contain even small quantities of organic matter are not acceptable bearing materials. Test results for these soils have been given for reference only.

3-4. ENGINEERING INVESTIGATION

General

The foundations of a building or other structure cannot be properly designed nor can they be cost effective without the benefit of an adequate investigation of the soil beneath the site of the proposed construction. This investigation should be authorized during the preliminary layout and planning stage of the project. Unexpected adverse soil conditions could require change in the architectural design and could also have a considerable effect on the cost projection of the project. The extent of this investigation must be tailor-made for the particular project. The responsibility for determining the extent and details of this investigation is normally delegated to the engineer who will ultimately be in charge of the foundation design. In any given project the engineering specifications may require the following work:

1. Test borings, including the recovery of undisturbed samples of soil and rock, depths to bedrock, contours and the preparation of drawings and descriptive logs of the work.
2. The installation of a perforated pipe in one or more of the bore holes and the monitoring of the depth to ground water for an extended period of time.
3. Field tests, as required to verify compaction to the specifications.
4. Laboratory analyses of undisturbed samples of soil and rock.
5. A report by the soils engineer, which will include the results of field and laboratory tests, and recommendations regarding bearing pressure and general construction procedures.

All work should be performed by a soils engineer or testing laboratory under contract with the owner. This company must work closely with the project architect and engineer to ensure compliance with the intention of this work. The contract specifications should include a procedure whereby this work can be changed in scope or in detail if conditions uncovered during the progress of the work indicate the need for such a change. This is particularly important in the case of the test borings because the exact number of holes and the total length of drilling can only be estimated until the work is actually being performed. Unit prices based on the addition or deletion of work can readily be agreed upon by contractor and owner before signing the contract.

The results of this investigation will give the project architect and engineer a detailed description of the site and will identify any unusual conditions which may require the use of special design or construction techniques. Included in this description will be recommendations as to allowable bearing pressures and information as to the existence and control of ground water.

Laboratory Tests

The laboratory testing of undisturbed samples of soil and rock is an integral part of any serious soils investigation and is used by the soils engineers to obtain the information required to make a valued judgement of the characteristics of the underlying soil or rock formations. The scope of information which can be obtained through laboratory testing is listed below. It is from this list that the engineer will determine the extent of the work to be performed under the basic soils contract.

For all soils:

1. Identification, in accordance with USCS
2. General characteristics
3. Unit weight
4. Porosity
5. Void ratio
6. Water content

For sands and gravels:

7. Angle of internal friction
8. Relative density

For silts and clays:

9. Unconfined compression strength
10. Cohesion
11. Sensitivity
12. Angle of internal friction (mixed-grained)
13. Shear strength

For expansive soils (in addition to those required for silts and clays):

14. Liquid limit
15. Plastic limit
16. Shrinkage limit
17. Plasticity index
18. Unrestrained swelling test
19. Swelling-pressure test

For rock:

20. Identification
21. General characteristics
22. Fissures, joints and cracks
23. Surface or joint weathering
24. Unconfined compression strength
25. RQD (rock quality designation)

3-5. TEST BORINGS

General Considerations

Test borings are holes drilled into the ground for the purpose of exploring the soils and rock formations that underlie the site of a proposed building. Test borings should normally be placed in a square grid measuring approximately 50 feet on a side, and the grid should extend approximately 25 feet beyond the extremities of the building. Borings should extend into the earth approximately 20 feet below the intended bearing elevation of the footings unless refusal is encountered at a higher elevation.

The procedures used in the making of test borings vary, depending on the kind of information required and whether the drilling is done in soil or in rock. Borings in soil are called earth borings. Borings in rock are called core borings.

3-6. EARTH BORINGS

An earth boring is a method of soils exploration in which a small hole is drilled into the ground for the purpose of recovering samples of soil for visual or laboratory examination. Samples are obtained by driving a sampling device into the undisturbed earth beneath the bottom of the hole. After the device has been filled with earth it is brought to the surface, and the sample recovered. Samples can consist of undisturbed soil or disturbed but representative soil depending upon the method of procurement.

In order to determine the variation in soil with increased depth, specifications usually require samples to be recovered typically at 5-foot intervals and at obvious changes in strata.

There are three methods by which earth borings are commonly made.

Method 1

In this method a section of 3- to 4-inch diameter hollow pipe called a casing is driven into the ground by a falling weight. By adding sections and continuing to drive, the pipe can be extended into the ground to almost unlimited depth. After the casing is driven to the depth at which it is desired to recover a sample, pressurized water is used to force the soil from the inside of the casing, thereby creating an open hole. The flow of water must be carefully controlled so that the soil below the casing will be disturbed as little as possible.

A sampling device is then lowered into the open hole and forced into the soil beneath. Because the hole is cased, this method of sample procurement is applicable to all kinds of soil. After the sample has been obtained and brought to the surface the casing is driven to a new depth from which a new sample is obtained. This process is repeated until the boring has reached the depth required by contract or amendment.

Method 2

In this second method, a continuous flight auger is advanced into the ground with a rotary drill, as illustrated in Figure 3-5. During this process the flights act as a screw conveyor and brings the spoils to the surface where they may be examined and then disposed. These spoils are very disturbed. They do, however, give a generalized overview of the soil and, because of their orientation with respect to the auger blades, they also give insight as to the variation in soil throughout the auger depth.

It may or may not be possible to procure earth samples by the use of this method. This can usually be determined by examining the spoils as they are brought to the surface. Soils with strong cohesive properties will remain stable when the auger is lifted out of the ground. This will result in an open hole. The sampling device can then be inserted through the open hole and a sample recovered. Note that the side walls of this hole may start to decay at any time. The sampling work, therefore, should proceed expeditiously.

Soils exhibiting granular characteristics will collapse into the hole as the auger is withdrawn. Samples in granular soils cannot be obtained.

Method 3

This method is similar to Method 2 in that a continuous flight auger is used. The primary difference is that this auger is constructed with a hollow stem. After the auger has been advanced to the depth at which a sample is required, the hollow stem is cleared of soil by pressurized water. This stem provides an open hole

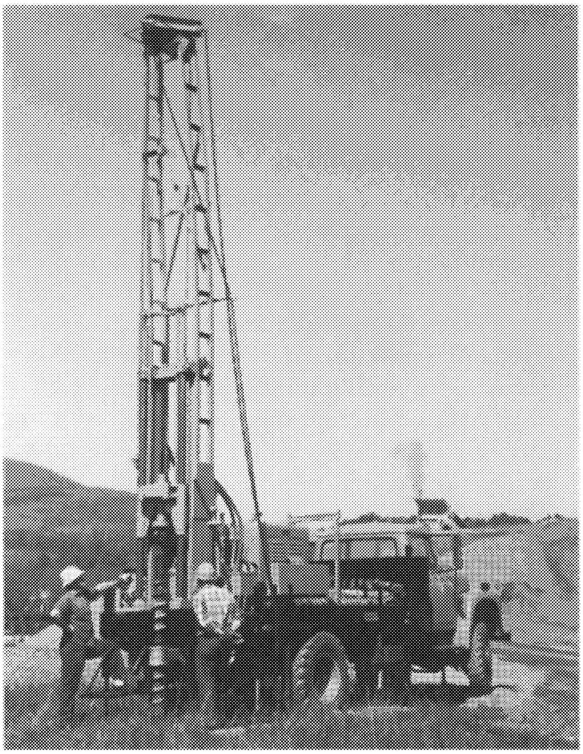


FIGURE 3-5. A rotary drill advancing a continuous flight auger. [Ref. 1]

for access and protection against earth cavein as did the casing of Method 1. The sampling device is then inserted through the stem into the undisturbed soil beneath and the sample is taken.

This method incorporates the best of the other two methods in that the screw conveyor effect brings spoils to the surface for visual examination, while the hollow stem permits samples to be taken in any kind of coarse or mixed grained soil.

One disadvantage to this method of augering is that a larger diameter auger is required to provide for the hollow stem. This requires the use of heavier machinery with a corresponding increase in cost.

Representative Samples

Representative samples, frequently called disturbed samples, can be obtained from all kinds of soils, regardless of composition. This kind of sample is usually obtained with a device called a split tube sampler, an example of which is shown in Figure 3-6.

The split tube sampler is designed to open into two halves, thereby exposing the recovered sample. These samples are representative of the material found at

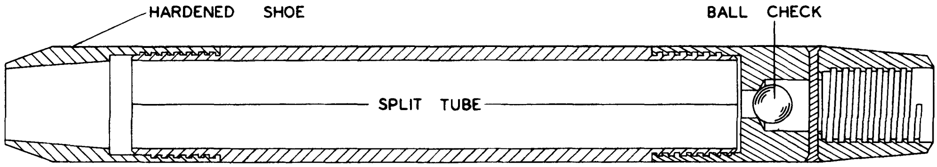


FIGURE 3-6. A split tube sampler, used for obtaining disturbed samples of granular and cohesive soils. [Ref. 1]

the bottom of the hole, but they do not represent the soil in situ. These samples, therefore, are of limited value for cohesive soils and are used primarily for the general identification of the soil. Their real value is in the recovery of granular soils where laboratory tests do not require undisturbed samples.

Split tube samplers are available in several different sizes. One of the more frequently used samplers has a 2″ outside diameter, a 1-3/8″ inside diameter, and a clear length of 18″ within the barrel. In all probability the main reason for the popularity of this particular sampler is that it is one of those specified for use in the standard penetration test, described later in this article.

Undisturbed Samples

Undisturbed samples are required for the laboratory analysis of soils having predominantly cohesive characteristics. Undisturbed samples may be obtained with the use of either of the following samplers:

1. The first is a modified version of the previously illustrated split tube sampler. This sampler is altered to house a thin wall liner. The sample is received in the liner which is then removed from the split tube for sealing and transportation to the testing laboratory.
2. The second is a thin wall tube sampler, a popular version of which is the Shelby Tube pictured in Figure 3-7.

Thin wall tube samplers have steadily gained in popularity since their development, and are now considered the industry standard for obtaining undisturbed samples. Different sizes of thin wall tube samplers are available. One of the more popular tube samplers has a 3″ outside diameter, a 2-7/8″ inside diameter and a clear length of 18″ within the barrel. These tubes are usually made of 16-gauge, seamless steel tubing.

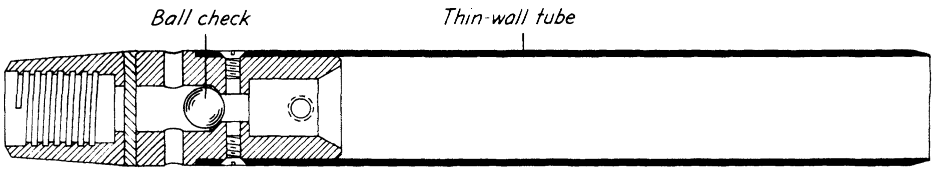


FIGURE 3-7. A thin wall tube sampler, used for obtaining undisturbed samples of cohesive soils. This sampler is frequently called a Shelby Tube. [Ref. 1]

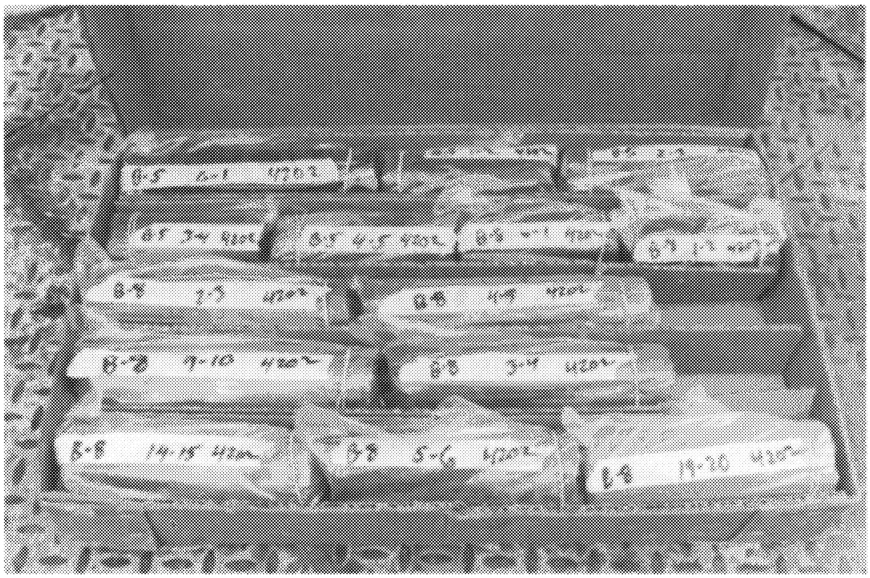


FIGURE 3-8. A box of undisturbed soil samples marked and ready for shipment.

With each kind of sampler it is important to protect the undisturbed samples from loss of moisture. This is accomplished by cutting off several inches from each end of the sample and filling the ends of the tube with hot wax or paraffin. Each sample is then carefully marked and boxed for shipment to the testing laboratory, as shown in Figure 3-8. Samples can also be transported in a dry-ice refrigeration box that slows down the migration of fluids, when such sophistication is required.

Standard Penetration Test

The standard penetration test, frequently abbreviated as SPT, is really a series of individual tests that are performed during the test boring operation. The purpose of this test is to provide insight as to the quantitative and relative strengths of the soils occurring at various depths throughout the site. The terminology usually associated with the resistance to penetration as indicated by this test is given in Table 3-2.

In this test, a split tube sampler is driven into the ground below the bottom of the bore hole using either of the two following combinations of driving energy and size of sampler:

Weight of Hammer	Height of Free Fall	Outside Diameter of Sampler
300#	18″	2-1/2″
140#	30″	2″

TABLE 3-2. Correlation between Soil Resistance to Penetration and Blow Count *N* [Ref. 20]

Sand (fairly reliable)		Clay (not as reliable)	
Relative Density	*N*	Consistency	*N*
Very loose	0–4	Very soft	< 2
Loose	4–10	Soft	2–4
Medium	10–30	Medium	4–8
Dense	30–50	Stiff	8–15
Very dense	>50	Very stiff	15–30
		Hard	>30

Note: The blow count *N* is taken from the boring log.

In driving, a record is kept of the number of blows (*N*) required to advance the sampler a measured distance of 12 inches. When making this test, the soil directly below the hole may be somewhat disturbed, thereby invalidating blow count readings taken within that height. To better ensure the validity of the readings, the sampler is usually driven a distance of 18″ below the bottom of the hole but readings of the number of blows are only recorded for the last twelve inches of driving.

N values serve an important function in determining safe values for bearing pressure in the design of spread footings, and for skin friction in the design of piles and piers. There are two instances, however, in which *N* values must be modified before their use. One relates to the occurrence of ground water and the other to release of overburden. These instances will be discussed in Article 3-7.

N values also serve an important function in providing approximate values for the angle of internal friction of coarse grained and mixed grained soils. These values are indicated in Figure 3-9.

Water Table, Perforated Pipe

The depth at which ground water is encountered is of great importance to any soils exploration. There are several reasons for this:

1. The blow count *N*, as described in the standard penetration test, is a major player in determining the strength of a soil in bearing, shear and in the control of settlement. The presence of ground water will require a modification to this blow count.
2. All occupied parts of the building below the water table must be waterproofed, and must be designed to resist the lateral pressure exerted by the ground water.
3. A high water table or deep excavation could cause serious problems with buoyancy, even to necessitating a change in the architectural design of the

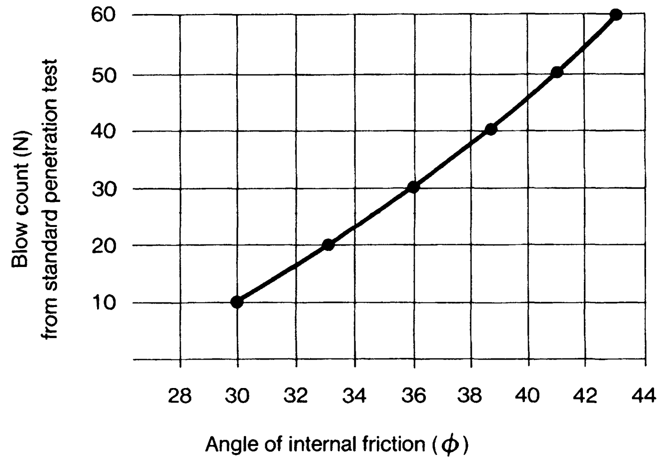


FIGURE 3-9. Approximate correlation between the angle of internal friction and the blow count N, for coarse grained soils. [Ref. 16]

building. Refer to Appendix F for information regarding the phenomenon of buoyancy.

The water table may fluctuate dramatically from season to season and also from drought to flood. In order to monitor the water table over an extended period, a perforated pipe should be installed in at least two of the holes left by the test borings. These pipes should extend to the bottom of the hole and should be capped with an easily removable screw cap. With this arrangement the depth to the water table can be monitored throughout the design stage of the project. Changes in design, if indicated by changes in the water table, can be made at this time.

3-7. MODIFICATIONS TO THE BLOW COUNT N

Modification Due to Ground Water

Ground water is what you find when you dig a well. The water table is the elevation of the surface of this water. The depth (D_w) is the distance to the water table measured from grade. When speaking of ground water, we are usually referring to the actual water and to the effect that it may have on adjacent or immersed structural elements. Water table is the term more frequently used in referring to an elevation or a depth.

For equal loading a footing bearing on saturated sand will settle more than a footing on dry sand. Subject to the level at which the water table occurs, the N values taken from the boring log should be modified by a correction factor (C_w).

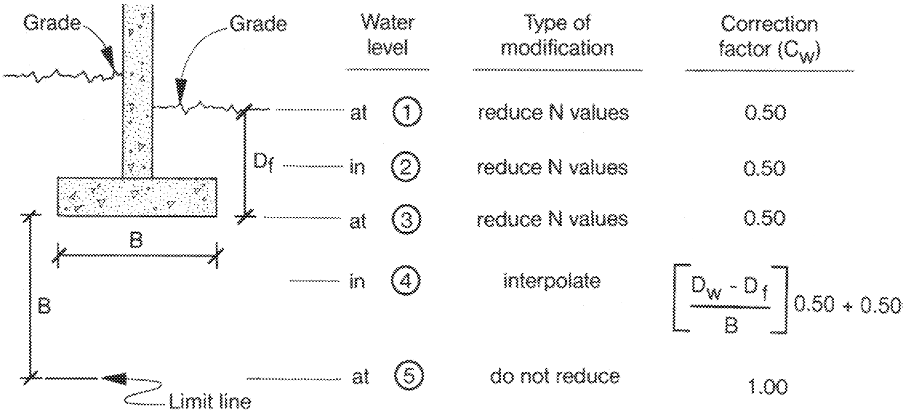


FIGURE 3-10. Blow count modification factor C_w, due to ground water, where the depth to ground water, D_w, is measured from grade.

The effect of this modification will be to increase the required size of footing. Only those blow counts that occur between the water table and an arbitrarily chosen limit line should be reduced. The limit line is typically located at a distance beneath the footing equal to the width of the footing. This condition, and the modifications required by it, are indicated in Figure 3-10. Note that this modification only applies to soils whose dominant fraction is sand.

Modification Due to the Release of Overburden

Blow counts quantitatively indicate the density of a sandy soil at the time when the borings are made. Blow counts normally increase with depth, thereby indicating an increase in density with a corresponding increase in bearing capacity and resistance to settlement. During the construction of the building, however, it is quite possible that the natural grade may be permanently lowered by reason of site development, or by substantial excavation required for basements or other substructures. In either event the overburden which existed at the time of the borings will not exist at the time of construction. When the restraining weight of the overburden is reduced the soil will rebound slightly, and there will be a corresponding reduction in density. This reduction in density can be numerically anticipated by multiplying the blow counts by a correction factor (C_N), as expressed in Formula (3-1). In this formula (p_0) is the weight of overburden above the elevation at which the blow count is to be modified.

$$C_N^{a} = 0.77 \log_{10}\left[\frac{40000}{p_0}\right] \leq 1.00 \text{ Max}^{b} \text{ [Ref. 16]} \qquad (3\text{-}1)$$

a. See Table 3-3 for values where $p_0 \geq 2000$ psf
b. Limiting value as recommended by the author: when $p_0 < 2000$ psf, no correction shall be made for the release of overburden

TABLE 3-3. Blow Count Modification C_N Due to Release of Overburden

Overburden p_o – psf	C_N[a]
2,000	1.00
3,000	0.87
4,000	0.77
5,000	0.70
6,000	0.63
7,000	0.58
8,000	0.54

[a] Values in accordance with Formula (3-1).

As the density of the sand is decreased when overburden is removed, it is also true that the density of the sand will be increased if overburden is added, as could occur by reason of the architectural design of the building. A corresponding increase in bearing capacity and resistance to settlement is therefore theoretically indicated. It is the general practice, however, to ignore the resultant increase in soil bearing capacity.

3-8. CORE BORINGS

In certain situations, it becomes necessary to carry the foundations down to bedrock. This may be due to engineering requirements or to the architectural design of the building. The usual reasons are as follows:

1. There is no satisfactory bearing material above the bedrock.
2. The weight of the building is so massive that adequate resistance in bearing can not be provided except by bedrock.
3. Anchorage of the building into bedrock is made necessary due to uplift caused by the overturning effect of wind or earthquake or to uplift caused by hydrostatic pressure.
4. The excavation required for construction of the building is so deep that it comes close to the surface of the bedrock or even extends into the bedrock.

When the foundations of a building bear on bedrock, it is necessary to determine the physical characteristics of the rock upon which the building will bear. This is accomplished by the procurement of rock samples through a process of core borings. The same machinery that was used for the earth borings is used for the core borings. The procedure is also similar in that a hollow casing is forced down through the soil while the soil within the casing is periodically removed. This casing, however, must be extended to refusal, defined as the point at which continued driving of the casing does not result in further penetration. A drill rod, fitted with a noncore recovery drill bit, is then used to extend the hole down into the rock to a point just above where the sample is desired. The drill rod is then

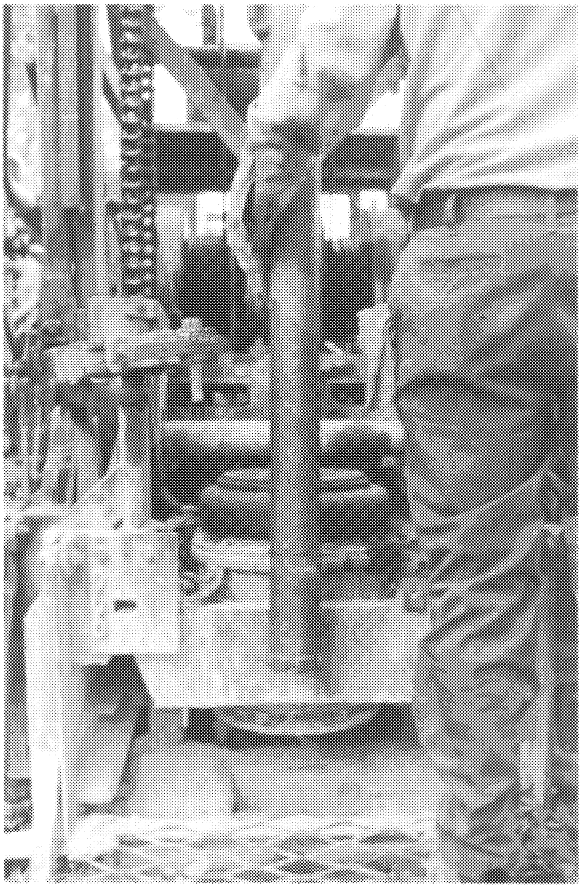


FIGURE 3-11. Drilling with a diamond core bit, for obtaining samples of hard rock.

brought to the surface and refitted with a core recovery barrel to which is attached a diamond core bit. This bit is drilled into the rock for the depth of the core barrel. The core is then recovered and brought to the surface. The equipment used in the procurement of core borings is illustrated in Figure 3-11.

Core barrels and diamond core bits are readily available in a variety of different designs and sizes. Proper selection depends primarily on the character of the rock and the difficulty encountered in obtaining the samples. A common type of diamond core bit is depicted in Figure 3-12. Although there are different types of core barrels and diamond core bits, their diameters, and therefore the diameter of the recovered samples, have been standardized by the industry. The more frequently used diameters are given in Table 3-4.

The function of the diamond core bit is to cut an annular ring into the rock for a depth of five feet. The five foot depth is used because this is the length of sample required for future rock quality evaluation. A cylindrical core of rock is

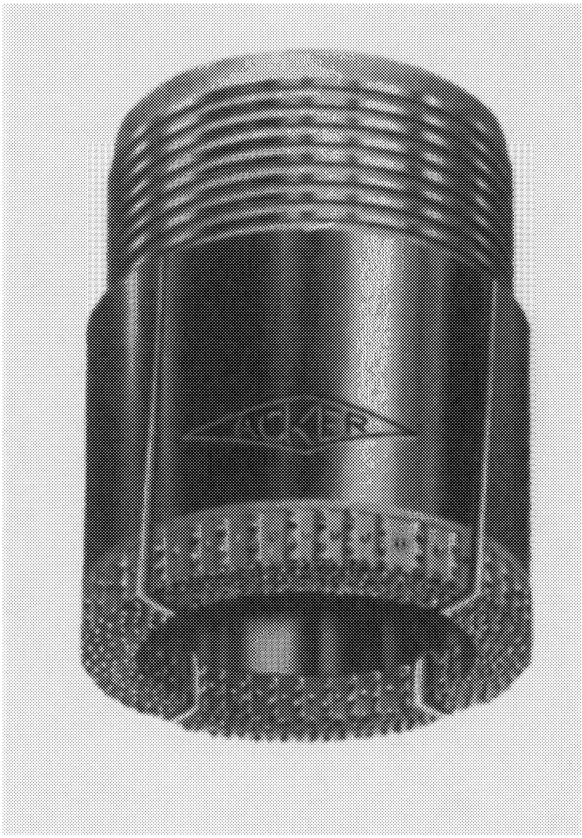


FIGURE 3-12. Enlarged view of a diamond core bit. [Ref. 1]

TABLE 3-4. Standard Core Diameters[a]

Outside Diameter of Core Barrel (in.)	Diameter of Core Recovered (in.)
1-1/2	7/8
1-7/8	1-1/8
2-3/8	1-5/8
2-15/16	2-1/8

[a] For *W* Group, *M* Design, by the Diamond Core Drill Manufacturers Association.

FIGURE 3-13. A typical core box, used to transport rock cores.

produced by the cutting action of the annular ring and is retained within the core barrel. This core will have an overall length of five feet but may be broken into several smaller pieces, some of which may even be badly chewed up. After the core barrel is brought to the surface and opened, the pieces of rock are placed in a core box, as shown in Figure 3-13.

It should be noted that it is very easy to inadvertently get the core turned upside down, especially if it is broken in small pieces. It must be remembered that the deepest part of the core comes out of the barrel first. The operators in the field should be cautioned to make sure that the core is correctly labeled as to top and bottom.

Core samples are ultimately delivered to the testing laboratory, where the strength and quality of the rock is evaluated, and bearing pressures are established. The testing and evaluation of rock is discussed in Chapter 14.

3-9. TYPICAL TEST BORING LOG

The log of a typical test boring is shown in Figure 3-14. The information shown on the log and described herein is representative of the type of information found on most test boring logs.

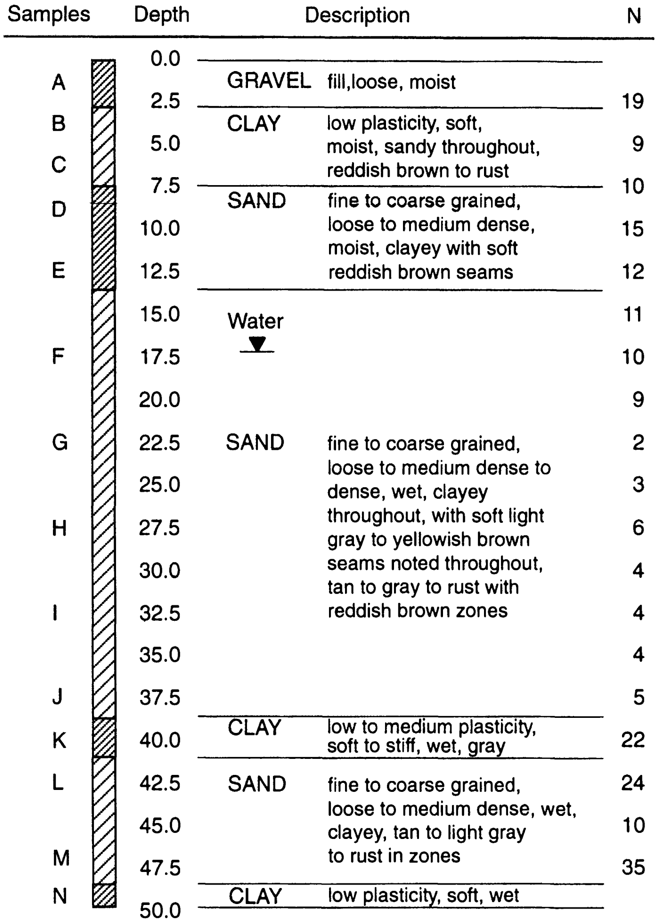


FIGURE 3-14. Boring log of hole No. 8A2S-2958, Fort Polk. [Ref. 22]

1. The N values (blow count) taken from the standard penetration test along with the depth at which the blows were measured
2. The depth at which there was a pronounced change in the character of the soil, the general classification of the soil within each of the different stratas, and a detailed description of the physical properties of the soil, as identified by sight, touch, and smell
3. The depth and designation of representative samples obtained during the test for further examination by others
4. The depth of ground water, if encountered
5. Elevation of bedrock, if borings were extended to that depth
6. Properties and characteristics of bedrock, when required

The indicated test boring was made by the Army Corps of Engineers for a Consolidated Supply Facility located at Fort Polk, under the date 9/19/88. Certain miscellaneous information has been omitted for brevity.

3-10. GEOLOGIC DESCRIPTION OF SITE

Test borings show only those subsurface features that occur at the immediate location of the boring. When borings are purposely laid out on a grid it is possible to obtain an overall description of the site by drawing a section through the various grids. One such section, taken from a project of which the author was the design engineer, is illustrated in Figure 3-15. The following information will assist in interpreting the contents of this figure.

1. *F* = Miscellaneous fill
   C = Medium to stiff brown and gray silty clay and clayey silt, with traces of fine sand and occasional organic material
   S = Medium compact to very compact brown and gray silty sand and gravel, with traces of silt
   B = Bedrock, relatively sound, slightly fractured, slightly weathered gray gneiss

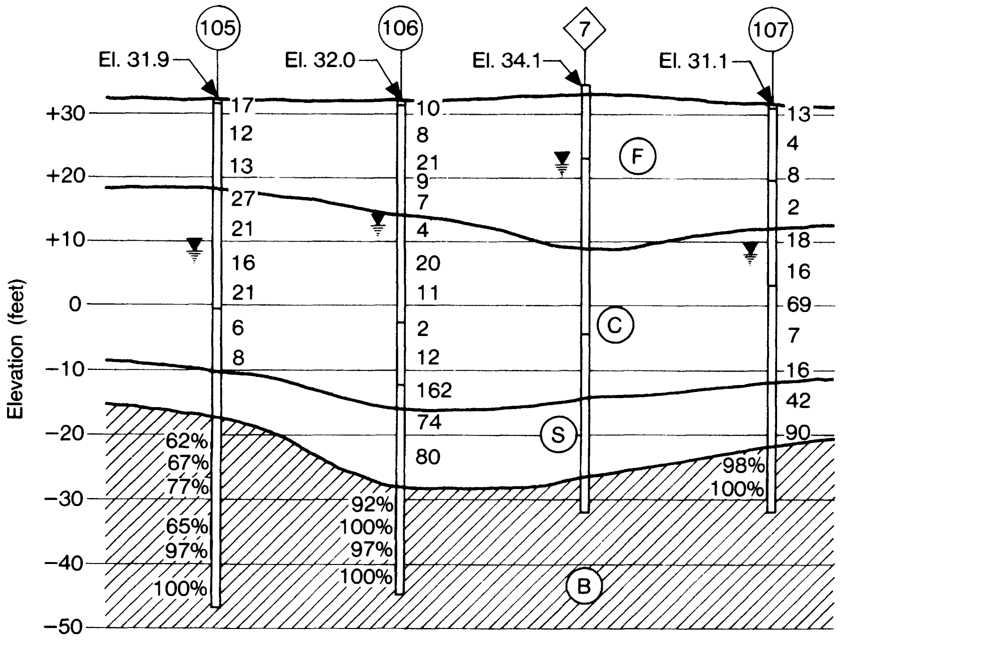


FIGURE 3-15. A geologic section taken through the site of one of the author's projects.

2. Numbers shown to the right of each boring are the blow counts recorded at those elevations during a standard penetration test; those to the left are the percentage of core recovery.

3-11. SAMPLE PROBLEMS

Example 3-1

Required: To determine the effect of ground water on the blow counts recorded during a standard penetration test.

Given: A 12-foot-square footing bears on a coarse grained soil of increased density with depth. The bottom of the footing is 4 feet below grade. The depth to ground water is 6 feet. Blow counts as given in Figure 3-16.

Refer back to Figure 3-11. Ground water occurs in level 4; therefore:

$$C_w = \left[\frac{6-4}{12}\right] 0.50 + 0.50 = 0.58$$

Only those blow counts that occur between the water table and the limit line are affected by ground water. For this particular example this will include all blow counts recorded from 6 to 16 feet below grade. This work is shown in Table 3-5.

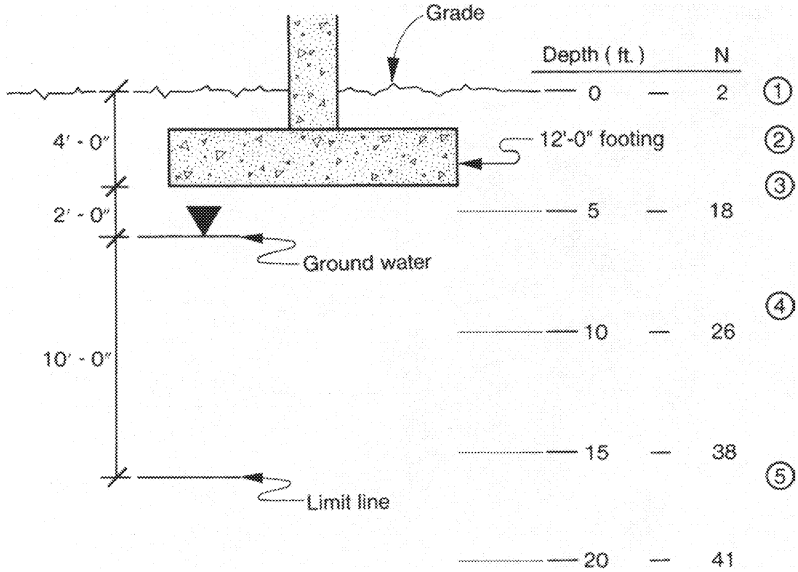


FIGURE 3-16. Example 3-1—Effect of ground water on blow count.

TABLE 3-5. Example 3-1—Blow Count Modification C_w

Depth	N^a	C_w	N^b
0	2	—	2
5	18	—	18
10	26	0.58	15
15	38	0.58	22
20	41	—	41

[a] Blow count taken from boring log.
[b] Blow count modified to account for the effect of ground water.

Example 3-2

Required: To determine the effect of future excavation on the blow counts recorded during a standard penetration test.

Given: The profile of the site as it presently exists and a profile of the intended future construction, as illustrated in Figure 3-17.

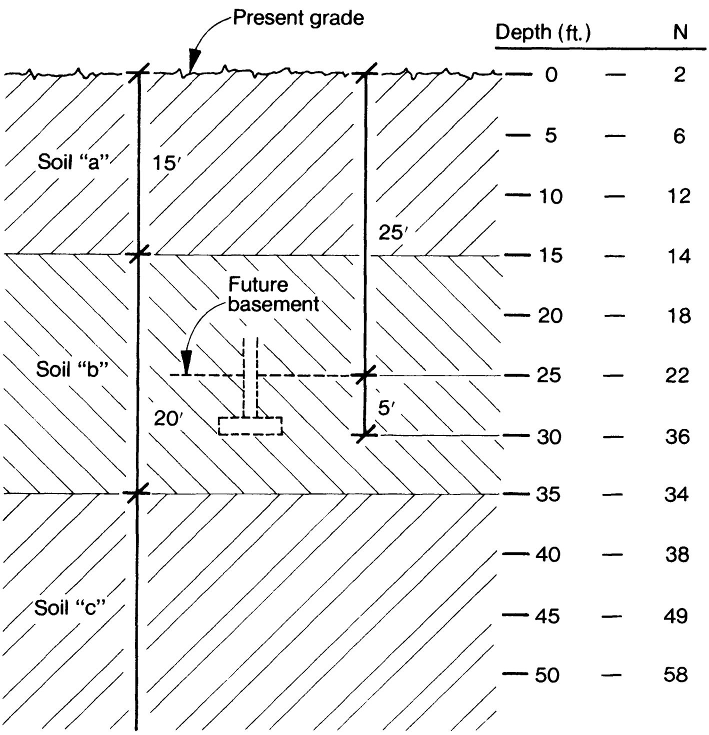


FIGURE 3-17. Example 3-2—Effect of excavation on blow count.

Soil properties are as follows:

Soil a—loose, gray sand, $\gamma = 115$ pcf
Soil b—medium sand with traces of gravel, $\gamma = 126$ pcf
Soil c—dense sand and gravel, mixed, $\gamma = 132$ pcf

Although blow counts from depth 0 to 25 experience a release in overburden, these blow counts do not enter into the design computations relative to footing capacity. They will, therefore, not be modified.

The depth of excavation required for construction of the footing is 30 feet, of which 5 feet will be replaced in order to construct the basement slab. The effective overburden to be removed, therefore, is 25 feet.

The weight of overburden $p_0 = 115 \times 15 + 126 \times 10 = 2985$ psf
From Table 3-3: $C_N = 0.87$
The use of this coefficient is shown in Table 3-6.

TABLE 3-6. Example 3-2—Blow Count Modification C_N

Depth	N^a	C_N	N^b
30	36	0.87	31
35	34	0.87	29
40	38	0.87	33
45	49	0.87	43
50	58	0.87	50

[a] Blow count taken from boring log.
[b] Blow count modified to account for the release of overburden.

4

Shear Strength of Soils

4-1. INTRODUCTION

Shear may be defined as the tendency of one part of a soil mass to slide with respect to the other. This tendency occurs on all planes throughout the soil mass. The singular plane of interest, however, is the plane of potential failure, called the plane of rupture.

Shear strength, as measured along this plane of interest, is the ability of the soil to resist the occurrence of a shear failure between the soils above and below the plane. All soils have the ability to develop strength in shear. Different soil groups develop this strength in different ways.

1. In sands and gravels this resistance is due to the physical interlocking of the soil particles, and is referred to as intergranular friction. Because this resistance is one of friction, its magnitude is a function of the particular details of the interlocking between the particles, and on the pressure of contact acting normal to the plane upon which shear is being considered. This resistance, then, is analogous to that developed by two pieces of rough sand paper when pressed together.
2. In cohesive soils there is very little, if any interlocking between the particles. The resistance to shear developed by these soils is primarily due to cohesion, which may be defined as a molecular force of attraction between particles. This resistance is analogous to that which would be exhibited between two sticky surfaces.
3. In mixed grained soils the resistance is equal to the combined action of friction, provided by the granular fraction of the soil, and cohesion, provided by the cohesive fraction of the soil.

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

The magnitude of shear strength is of primary importance in the consideration of the following areas of foundation design:

1. Ultimate bearing capacity for spread footings
2. Frictional resistance for piles and piers
3. Stability of slopes
4. Determination of the lateral earth pressure exerted on a retaining wall

These considerations will be addressed in subsequent chapters.

Note: Shear strength, when related to soil, has many names. It is also referred to as shear capacity, resistance to shear, or friction.

4-2. THE COULOMB EQUATION FOR SHEAR RESISTANCE

The unit resistance to shear which may be developed along the plane of rupture by any soil can be determined analytically by the following equation, as proposed by C.A. Coulomb, a French scientist, in his theory on earth pressures, published in 1773:

$$s = c + p \tan \phi - \text{psf} \tag{4-1}$$

In which:

s = the unit resistance to shear developed by the combined action of cohesion and friction
c = that part of the resistance due to cohesion, and is attributable to the fine grained fraction of the soil
p = the pressure acting normal to the plane of rupture
ϕ = the angle of internal friction
$p \tan \phi$ = that part of the resistance due to friction, and is attributable to the coarse grained fraction of the soil

The unit resistance to shear, as determined by the Coulomb equation, is dependent upon the classification of the soil. Granular soils develop their resistance through friction, while clay soils develop theirs through cohesion. The resistance developed by a mixed grained soil will be the sum of the separate parts. The difference in resistance as a function of soil classification is illustrated graphically in Figure 4-1.

4-3. TESTS FOR SHEAR STRENGTH

The ability of a soil to support vertical loads and to resist the sliding effect of lateral loads is governed to a large extent by the shear strength of the soil. It is

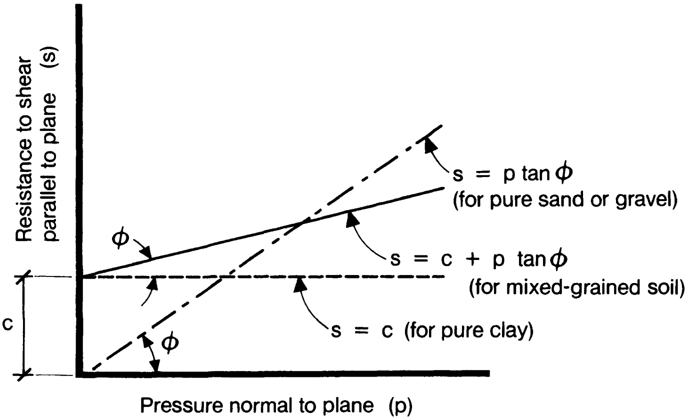


FIGURE 4-1. Graphic illustration of the Coulomb equation for shear resistance of different kinds of soil.

therefore important to determine accurately the shear strength of soils situated beneath and in close proximity to the proposed construction. There are several field and laboratory tests by which shear strength can be determined with reasonable accuracy:

1. Vane shear tests, taken in the field on undisturbed soil
2. Direct shear tests, performed in the laboratory
3. Triaxial compression tests, performed in the laboratory

4-4. VANE SHEAR TEST

The vane shear test is a field test whose use is limited to the testing of cohesive soils. It is unreliable for clays containing any appreciable amount of silt or sand, and cannot be used on coarse grained soils.

Vane shear testing, however, is a procedure that will provide immediate and accurate results when conducted properly by trained personnel using good equipment. Representative vanes and a schematic diagram of a test in progress are illustrated in Figure 4-2. Procedures for this test are governed by the following ASTM Standard:

ASTM Designation D-2573: Field Vane Shear Test in Cohesive Soil

This test requires the installation of a casing similar to that which would be used in a test boring. The soil that accumulates within the casing must be removed before proceeding with the work. The cleaning out process must be carefully performed, so as not to disturb the soil beneath. A clean-out auger is recommended for this purpose.

To perform this test, the vane is pushed into the soil below the casing, being

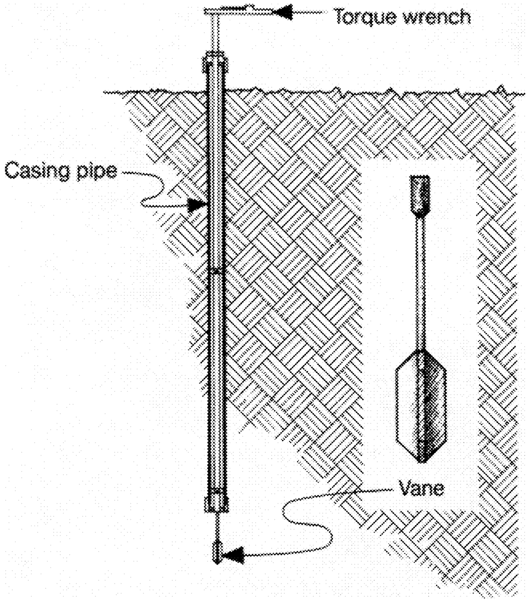


FIGURE 4-2. Vane shear apparatus. [Ref. 1]

careful to disturb the soil as little as possible. The vane is then slowly rotated with a very accurately calibrated torque wrench until the soil fails. The shear strength of that particular soil is then read off of a shear torque chart, which correlates the shear strength of the soil in pounds per square inch to the torque reading at the time of failure. There are separate charts for vanes of different diameters and lengths.

Because of its inherent accuracy and simplicity of operation, vane shear testing can be used as the primary source of the shear strength evaluation of a cohesive soil. Undisturbed samples tested in the laboratory can be used as a secondary source and as confirmation of the field test results.

4-5. DIRECT SHEAR TEST

The direct shear test may be performed in the laboratory on the following types soil:

a. Undisturbed samples of a cohesive soil
b. Representative (disturbed) samples of coarse or mixed grained soils, provided that the sample is compacted before testing to the density and void ratio of the soil it represents

This test is governed by the following ASTM Standard:

ASTM Designation D-3080: Direct Shear Test of Soils Under Consolidated Drained Conditions

This test will establish the variation in shear strength of the soil on a given plane as a function of the pressure applied normal to the plane. It will also determine the angle of internal friction and unit cohesion of the soil.

In this test a sample of soil of known area is held in a container which is split horizontally into two sections. The lower section of the container is held in position, while the upper part is subjected to a lateral shear force. During this test, the sample is also subjected to a force acting normal to the plane of shear. While this normal force is held constant, the shear force is steadily increased until failure occurs. The forces acting at the time of failure are recorded, and from these the normal stress p and the shear stress v are computed. This sample is then discarded, and the test is repeated several times using a new sample for each test. The results of these tests can then be plotted, using the normal stress on the abscissa and the shear stress on the ordinate. The resulting graph will approximate a straight line, and will correspond to one of the lines previously illustrated in Figure 4-1. Note that the magnitude of the angle of internal friction and the unit cohesion can be measured directly from this graph.

It is inherent to this test that the sample fail on a horizontal plane. For purely cohesive soils, the maximum induced shear will occur on this plane, but for granular and mixed grained soils the maximum shear will occur on some other plane. It should be remembered that the plane upon which the maximum shear is induced is not necessarily the plane upon which rupture will occur. The triaxial compression test, as discussed in the next article, is designed to permit the sample to fail along the plane on which the induced shear exceeds the capacity of the soil in resistance to shear, regardless of orientation.

4-6. TRIAXIAL COMPRESSION TEST

General Procedure

The triaxial compression test may be performed in the laboratory on the following types of soil:

a. Undisturbed samples of a cohesive soil
b. Remolded samples of a cohesive soil, provided that the void ratio and water content remain unchanged
c. Representative (disturbed) samples of coarse or mixed grained soils, provided that the sample is compacted before testing to the density and water content of the soil it represents

This test must be performed in accordance with the following ASTM Standard:

ASTM Designation D-2850: Undrained Compression Strength of Cohesive Soils in Triaxial Compression

Before testing, the sample is encased in a flexible membrane, usually made of rubber. The sample is then enclosed within a cylindrical device which permits the operator to subject the sample to an all-around confining pressure by the introduction of pressurized air or water into the chamber. The chamber is also constructed so as to allow an additional vertical pressure to be gradually applied axially to the sample.

The test is conducted by holding the all-around confining pressure constant while steadily increasing the overall vertical pressure applied axially to the sample. The general test setup and the pressures applied to the sample during this test are indicated diagramatically in Figure 4-3, in which:

p_3 = the all-around confining pressure
p_2 = the additional axial pressure
p_1 = the resultant vertical pressure = $p_3 + p_2$

The pressures occurring at the time of failure are recorded. The sample is then discarded and the test is repeated several times using a new sample for each test.

As previously noted, this test is constructed in such a way to allow failure to occur on the plane where the induced shearing stress exceeds the capacity of the soil in shear resistance. This plane of rupture, signified by the line AB, is identified

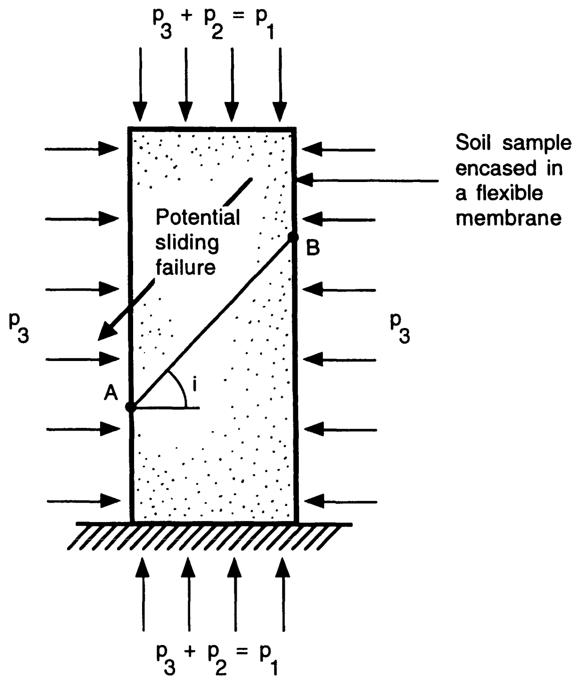


FIGURE 4-3. Triaxial compression test showing test pressures and assumed Plane of failure *AB*.

by the angle (i). This angle may be defined as the angle of rupture. The reader should note the similarity between this mode of failure and the one previously indicated in Figures 2-4 and 2-5.

Mathematical Analysis of the plane AB

The geometry of the reference plane and the forces acting on it are shown in Figure 4-4, the four parts of which represent the following:

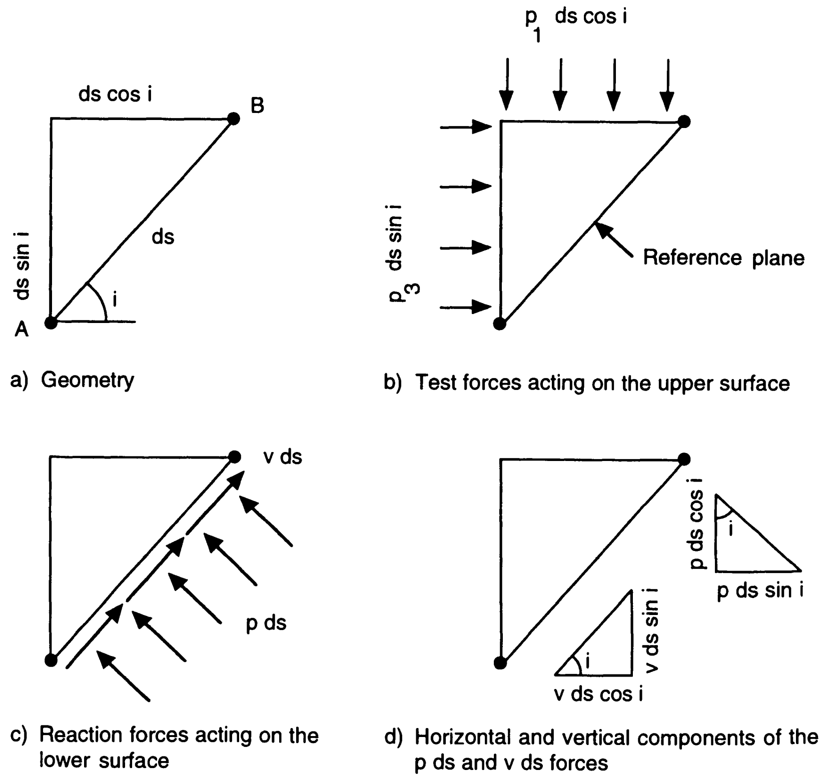


FIGURE 4-4. Criteria for the analysis of the reference Plane *AB*.

Figure 4-4(a). This indicates the geometry of the plane, assuming one unit of width into the plane. The lines specified, therefore, are representative of areas.

Figure 4-4(b). These are the external forces acting on the plane due to the test pressures p_1 and p_3. These forces act on the upper surface of the plane and tend to induce the sliding failure that was indicated on Figure 4-3.

Figure 4-4(c). These are the internal forces acting on the lower surface of the plane and developed by the sample in response to the induced sliding action. As long as these forces provide adequate shearing resistance, failure will not occur.

Figure 4-4(d). These are the horizontal and vertical components of the internal forces indicated in Figure 4-4(c).

Before failure, the forces acting on the plane must be in equilibrium, therefore:

$$\text{For } \sum H = 0 \qquad p_3 \text{ ds Sin } i + v \text{ ds Cos } i - p \text{ ds Sin } i = 0$$

$$\text{For } \sum V = 0 \qquad p_1 \text{ ds Cos } i - v \text{ ds Sin } i - p \text{ ds Cos } i = 0$$

In which:

p is the confining pressure, acting normal to the plane

and: v is the induced shear stress, acting parallel to the plane

Note that p and v are both resultant pressures, produced by the action of the pressure applied during the test program.

The above equations are then solved simultaneously to obtain formulas isolating p and v. The resulting formulas demonstrate the dependency of each value on the angle i.

$$p = 0.5\ (p_1 + p_3) + 0.5\ (p_1 - p_3) \cos 2i \tag{4-2}$$

$$v = 0.5\ (p_1 - p_3) \sin 2i \tag{4-3}$$

Pressure Variations as a Function of the Angle i

Numerical values of p and v can be computed using formulas (4-2) and (4-3). As an example of the method, values for p and v will be computed, assuming that $p_3 = 400$ psf and $p_1 = 2200$ psf.

With the pressures p_3 and p_1 being held constant, the values of p and v depend solely upon the angle i.

For purposes of illustration, assume that angle $i = 15°$:

$$p = 0.5\ (2200 + 400) + 0.5\ (2200 - 400) \cos 30° = 2079 \text{ psf} \tag{4-2}$$

$$v = 0.5\ (2200 - 400) \sin 30° = 450 \text{ psf} \tag{4-3}$$

New values of p and v are computed, using angle values of 30°, 45°, 60°, 75°, and 90°. The results are recorded in Table 4-1 and plotted in Figure 4-5.

The table and figure values indicate that the maximum shearing stress occurs on the oblique plane for which the angle i equals 45°. It must be noted, however, that this does not necessarily mean that the plane of rupture occurs at this angle. This will require further consideration.

TABLE 4-1. Computed Values of *p* and *v* on Reference Plane AB from Formulas (4-2) and (4-3)

$i°$	$2i°$	p	v
0	0	2200	0
15	30	2079	450
30	60	1750	779
45	90	1300	900
60	120	850	779
75	150	521	450
90	180	400	0

Note: Units of p and v are psf.

An examination of Figure 4-5 suggests that these points all lie on a circle having a radius of:

$$R = 0.5\ (p_1 - p_3)$$

and whose origin is at $p = p_3 + 0.5\ (p_1 - p_3) = 0.5\ (p_1 + p_3)$

It can be demonstrated mathematically that these points do, in fact, form a circle. It can also be shown geometrically that the central angle, when measured counterclockwise from the horizontal axis to the radial line intersecting the point (p, v), if equal to $2i$. The proof of these statements is given in Appendix H.

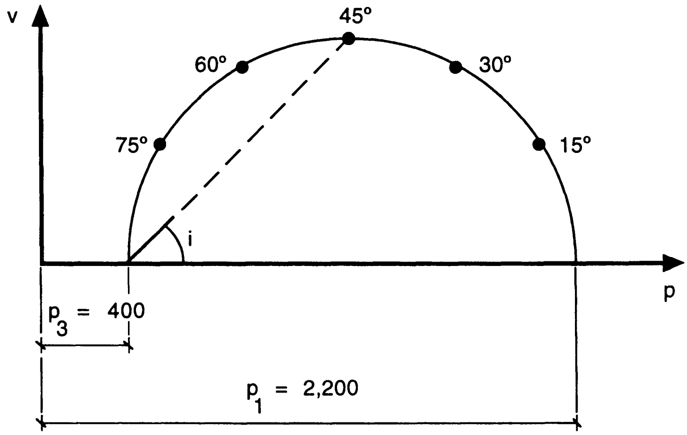


FIGURE 4-5. Locus of points (p,v) for various orientations of the Reference Plane *AB* as defined by the angle *i*.

4-7. MOHR'S CIRCLE OF STRESS

Purpose

Mohr's circle of stress is a procedure used in conjunction with the triaxial compression test for the purpose of determining the following values:

1. The angle of internal friction (ϕ) of the soil
2. The unit cohesion (c) of the soil
3. The unique value of the angle (i) that defines the plane of rupture
4. The normal pressure (p) and shear stress (v) induced onto the plane of rupture
5. Similar values for (p) and (v) induced onto any other plane of interest

This procedure does not lend itself to an analytic solution. All work is done graphically.

Construction of Mohr's Circle

Points A and B are laid out on the horizontal axis using the numerical value of the test pressures p_3 and p_2. Point C is marked midway between points A and B. Using point C as the center of a circle, draw a semicircle using a radius equal to 0.5 p_2. Mark a general point D, the point of maximum shearing stress E and the angles i and $2i$. This is shown in Figure 4-6.

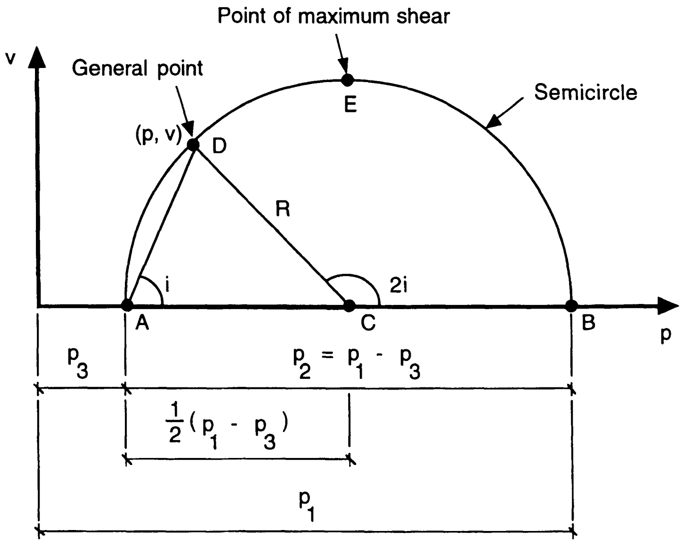


FIGURE 4-6. Construction of Mohr's circle of stress.

The horizontal and vertical coordinates of any general point D on the circle represent the normal and shearing pressures induced on the plane identified by the angle i.

The Use of Mohr's Circle

When plotted to scale, Mohr's circle enables the reader to graphically obtain the magnitude of the shear stress induced on any specified plane of interest as identified by an assumed angle i. Our usual interest, however, is in determining the unique value of i that identifies the actual plane of rupture, from which the shear induced at failure can be computed.

An examination of Figure 4-6 shows that the maximum shear occurs at point E on the oblique plane for which the angle i is 45°. Maximum shear, as noted previously, does not necessarily mean that the plane of rupture occurs at that angle. It will be seen that the angle of rupture is solely dependent upon the angle of internal friction of the particular soil being tested.

The angle of internal friction may be found by plotting the results of a series of tests relating to the same soil. This is shown in Figure 4-7, which provides the following information:

1. The angle of internal friction, which is defined by the line drawn tangent to the test circles
2. The cohesion, defined as the shear at which the tangent line meets the vertical axis
3. The orientation of the plane of rupture, as identified by the angle i
4. The normal pressure (p) and shear stress (v) induced onto the plane of rupture

The tangent line, as drawn in Figure 4-6, is representative of a mixed grained soil. The tangent line of purely granular soils and purely cohesive soils will have a different orientation. This was previously demonstrated in Figure 4-1.

4-8. NUMERICAL DETERMINATION OF THE ANGLE OF RUPTURE

The plane of rupture, as identified in Figure 4-3, is defined by the angle i, measured from the horizontal. The magnitude of this angle may be determined by referring to Test No. 3 in Figure 4-7 and applying the following identity:

$$2i = 180° - (90° - \phi) = 90° + \phi$$

$$\text{Therefore: } i = 45° + \frac{\phi}{2} \tag{4-4}$$

The plane of rupture, therefore, may be defined by the above angle.

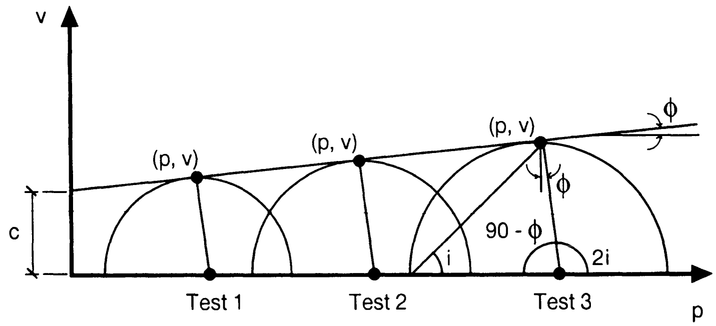


FIGURE 4-7. Plot of a series of Mohr's circles.

Note: In Article 9-5, the plane of rupture was defined by the angle (α), measured from the vertical. The sum of these two angles is 90°. Therefore, these different methods of angle identification are compatible.

A Word of Caution

In Figures 4-6 and 4-7, the point (p,v) identifies the normal pressure and shear stress induced on the plane defined by the angle i. In Figure 4-6, the point is a perfectly general point, and the angle does not necessarily define the plane of rupture. In Figure 4-7, however, the point (p,v) is intersected by the tangent line whose angle defines the angle of internal friction. The unique angle i that intersects this point of tangency is the true angle of rupture, and the stresses given are those that occur on the plane of rupture.

4-9. SAMPLE PROBLEMS

Example 4-1—Direct Shear Test

A direct shear test consisting of three separate tests was run on a mixed grained soil. The results of this test are as follows:

Test No.	Applied Normal Stress p (psf)	Resultant Shear Stress v (psf)
1	400	1435
2	600	1506
3	1000	1680

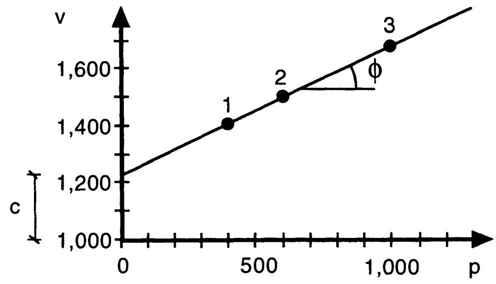


FIGURE 4-8. Example 4-1—Direct shear test.

Required: To determine the angle of internal friction and the cohesion of this soil, using the procedures described in Article 4-5.

The test results are plotted graphically on Figure 4-8. From this figure the following information may be obtained graphically:

$$\phi = 24° \qquad c = 1230 \text{ psf}$$

More accurate values could have been obtained analytically, but one of the advantages of this test is that it lends itself so easily to the graphic solution.

Example 4-2—Mohr's Circle of Stress

A triaxial compression test consisting of three separate tests was run on a mixed grained soil. The results of this test are as follows:

Test No.	Confining Pressure p_3 (ksf)	Additional Axial Pressure p_2 (ksf)	Resultant Axial Pressure p_1 (ksf)
1	1.0	3.7	4.7
2	2.0	4.9	6.9
3	4.0	7.0	11.0

Required: To determine the angle of internal friction and the cohesion of this soil, using the procedures described in Article 4-7.

The test results are plotted graphically in Figure 4-9, using the concept of Mohr's circle of stress. From Figure 4-9, the following information is readily obtained by scale:

$$\phi = 20° \qquad c = 0.9 \text{ ksf}$$

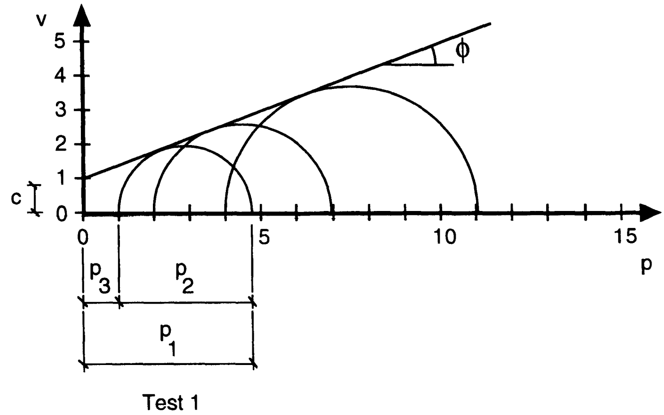


FIGURE 4-9. Example 4-2—Mohr's circle of stress.

If it were additionally required to determine the angle of rupture, the angle i could be measured from any one of the three circles, or it could be calculated by using Formula (4-4). Using either method, $i = 55°$.

5

Allowable Soil Bearing Pressure

5-1. GENERAL DESIGN CONSIDERATIONS

This chapter provides design criteria for shallow foundations, commonly referred to as spread footings. Design criteria for deep foundations (piles, piers, and caissions) is addressed in Chapter 8. Details relative to the construction of spread footings is given in Chapter 7.

It is very difficult to predict accurately the way in which a footing will respond to the loads imposed upon it. There are many intangibles, not the least of which is that an earth mass is rarely homogeneous. Even the thick beds of sand and gravel found along most of the coastlines are not truly homogeneous. Soils are almost invariably mixed-grained with each material imparting its own individual characteristics to the mass. Soils may also be layered with each layer having different characteristics and responding differently to the loads to which it is subjected.

Other intangibles include the effects of intermittent loading due to live load, dynamic loading due to wind or earthquake, the possible rise or fall in the water table over a number of years and the potentially destabilizing effect of future construction in close proximity to the site.

The engineer in charge of the design of the foundations must recognize the many intangibles involved. Foundation design is an art as well as a science. Foundations cannot be adequately designed nor can they be cost effective unless the engineer can combine the proper mixture of theory, experience and intuition into all his work.

The first step in the design of a spread footing is to determine the allowable bearing pressure for which the footing shall be designed. This is the pressure that will exist on the contact surface between the footing and the soil which

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supports it. The determination of this allowable bearing pressure requires consideration of the following two situations:

1. The literal failure of the soil to support the imposed load, which results in the footing breaking into the ground, as discussed in subsequent articles.
2. Settlement which, when excessive, will cause severe damage to the structure itself and to any elements attached to it (Considerations of settlement will be discussed in Chapter 6.)

5-2. SOIL BEHAVIOR AT ULTIMATE BEARING CAPACITY

The soil beneath a footing will fail when the load to which it is subjected exceeds the ultimate bearing capacity of the soil. In this kind of failure, as illustrated in Figure 5-1, the footing literally breaks into the ground. This kind of failure is very dangerous because it occurs suddenly, and its effect will be felt throughout the structure. Such a failure can result in catastrophic damage and the possible loss of life.

Soil failure is the result of displacement of the soil directly beneath the footing. This action is not completely understood, although different theories have been presented. There is general agreement, however, that the displacement is resisted by the combined action of shear and lateral passive pressure. When failure occurs, the footing will invariably lurch to one side. Note the falling in of the earth on one side of the footing and the pushing up of the earth on the other side.

It has been noted in Article 4-1 that the resistance to shear developed by a coarse grained soil is due to the actual interlocking and physical contact between the particles. Its value, numerically, is a function of the angle of internal friction of the particular soil. In fine grained soils there is no interlocking or physical contact between particles. Resistance to shear for these soils is a function of

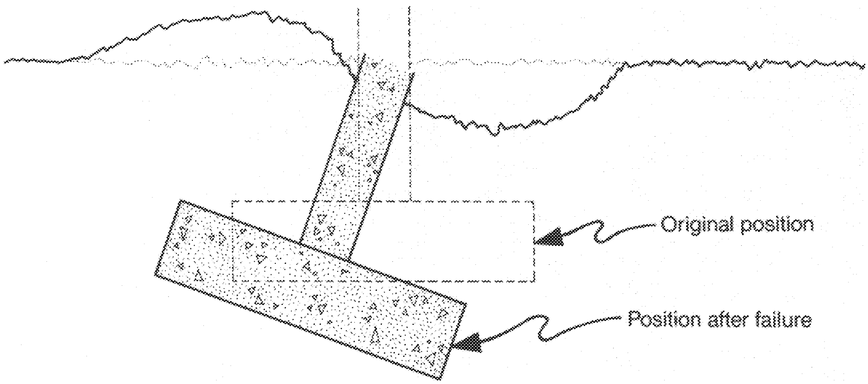


FIGURE 5-1. Footing failure due to breaking into the soil.

cohesion. In mixed grained soils the total resistance to shear is the sum of the resistances contributed by each of the different kinds of soil.

Resistance due to lateral passive pressure can be similarly described. In coarse grained soils lateral displacement can only occur by reason of a rearrangement of the individual grains of soil within the mass. In fine grained soils the displacement will occur by reason of a decrease in void ratio caused by the squeezing out of air and water from within the mass.

5-3. EQUATIONS FOR ULTIMATE BEARING CAPACITY

General Considerations

A comprehensive theory by which the response of soil to load could be quantitatively analyzed was first advanced by Karl Terzaghi, considered by many to be the father of modern soils engineering. The equations and bearing capacity factors that follow have their origin in the Terzaghi theory of soil analysis, but incorporate certain modifications as proposed by others, including Meyerhof, Hanson and Vesic, during the ensuing years. For those instances when different sources have proposed different modifications, there has been a tendency on the part of the author to take the more conservative approach. Experience has demonstrated time and time again that a conservative approach in the design of the foundations of a building results in sound, cost effective, and prudent engineering.

Footings on Mixed Grained Soil

Equations for the determination of the ultimate bearing capacity of a footing on a mixed grained soil are given below. The first term in these equations provides for the contribution of cohesion. The second term provides for the influence of surcharge, and the third term provides for the effect of soil density and shape of the footing.

Continuous footing:

$$q_d = 1.0\, cN_c + \gamma D_f N_q + 0.5\, B\, \gamma\, N_\gamma \tag{5-1}$$

Square footing:

$$q_d = 1.2\, cN_c + \gamma D_f N_q + 0.4\, B\, \gamma\, N_\gamma \tag{5-2}$$

Round footing:

$$q_d = 1.2\, cN_c + \gamma D_f N_q + 0.6\, R\, \gamma\, N_\gamma \tag{5-3}$$

TABLE 5-1. Shape Factors for Rectangular Footings [Ref. 13]

Length to Width ratio	a_1	a_2
1	1.20	0.42
2	1.12	0.45
3	1.07	0.46
4	1.05	0.47
6	1.03	0.48
Continuous	1.00	0.50

Rectangular footing:

$$q_d = a_1\, cN_c + \gamma D_f\, N_q + a_2\, B\, \gamma\, N_\gamma \tag{5-4}$$

In which:

q_d = the ultimate bearing capacity of the soil at the base of the footing (psf)
c = the cohesion of the soil (psf)
B = the width of a continuous footing, the lesser width of a rectangular footing or the side of a square footing (feet)
R = the radius of a round footing (feet)
a_1 and a_2 = shape factors for use with rectangular footings whose numerical values are given in Table 5-1
γ = the density of the in situ soil, obtained by direct measurement, guidelines of which are given in Table 2-1 (pcf)
D_f = the depth of the footing below the lowest adjacent grade (feet)
N_c, N_q, and N_γ = dimensionless bearing capacity factors, the values of which are numerically dependent on the angle of internal friction of the soil beneath the footing

The numerical value of these three bearing capacity factors can be computed from the formulas which follow, or can be taken directly from Table 5-2 or Figure 5-2:

$$N_q = e^{\pi \tan\phi}\,[\tan^2(45^\circ + \frac{\phi}{2})]$$

$$N_c = (N_q - 1)\cot\phi$$

$$N_\gamma = (N_q - 1)\tan(1.40\,\phi)$$

In the above formulas, ϕ is the angle of internal friction, as introduced in Article 2-12.

The angle of internal friction of a granular soil can only accurately be deter-

TABLE 5-2. Numerical Values of N_C, N_q, and N_γ for Shallow Foundations

Angle ϕ	N_C	N_q	N_γ
16	11.6	4.3	1.4
18	13.2	5.3	2.0
20	14.8	6.4	2.9
22	16.8	7.8	4.1
24	19.3	9.6	5.7
27	23.9	13.2	9.5
30	30.1	18.4	15.7
33	38.7	26.1	26.2
36	50.7	37.8	44.5
39	67.9	55.9	77.3
42	93.7	85.4	139.3
45	133.9	134.9	262.7

Note the disproportionate increase in the values for N compared to those for ϕ.

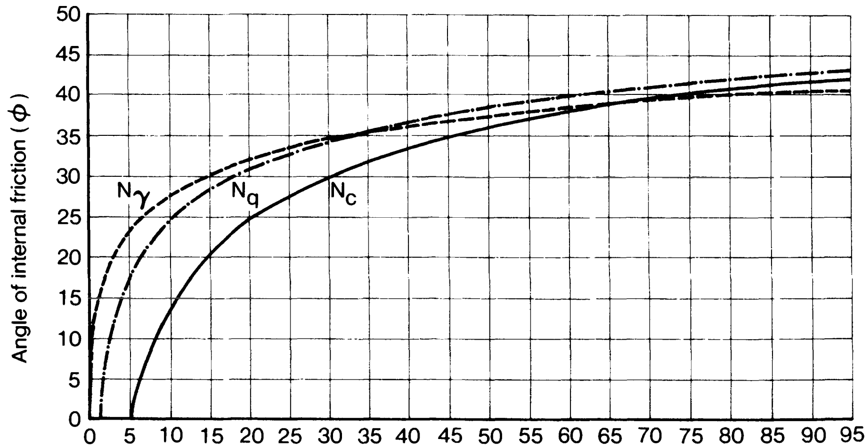


FIGURE 5-2. Bearing capacity factors for shallow footings as a function of the angle of internal friction.

mined by laboratory analysis. There are times, however, when approximate values are satisfactory. This would be true in the case of a feasibility study or even in a preliminary design. For a mixed grained soil whose characteristics are similar to those of a granular soil, there is a reasonably reliable correlation between the numerical value of the angle of internal friction and the blow count N, as recorded during a standard penetration test. This correlation is given in Figure 3-9. For a purely granular soil the angle of internal friction can be approximated from Table 2-4.

The numerical value of cohesion can be determined by laboratory analysis when accurate values are required. Under normal circumstances, however, experience indicates that the cohesion may be assumed equal to one-half of the unconfined compression strength q_u. Accurate values for q_u can be determined in the laboratory by means of an unconfined compression test or its value can be approximated by correlation to blow count, as given in Table 8-3.

When applying Equations (5-1) through (5-2) it should be noted that the density used in the second term of the equations should theoretically relate to the soil above the footing, as this is what causes the surcharge. The density in the third term should relate to the soil below the footing since this is directly related to bearing. Generally, there will be little difference between these two densities. Therefore, in the interest of simplifying the work it is customary to use the lesser value. This is on the side of safety.

Footings on Granular Soil

Pure sands and gravels are granular materials that have no cohesion. The resistance to penetration of the footing depends, therefore, solely on the physical contact and degree of interlocking between the particles. When computing the ultimate bearing capacity of a footing bearing on a purely granular soil, the first term in the general equations, that which enumerates the effect of cohesion, should be deleted. The second and third terms should remain.

Footings on Cohesive Soil

Pure clay and plastic silt are materials whose particles do not interlock. Resistance to soil failure for such materials depends solely on cohesion, which is a characteristic of all clays and of the finer fractions of silts. The angle of internal friction of a purely cohesive soil is zero. When computing the ultimate bearing capacity of a footing bearing on a cohesive soil, the third term in the general equation, which includes the effect of the soil density, should be deleted. The first and second terms should remain. Bearing capacity factors for a cohesive soil are as follows:

$$N_c = 5.14 \qquad N_q = 1.0 \qquad N_\gamma = 0$$

> *Note:* In computing the value for N_c the equation becomes discontinuous at $\phi = 0$, at which point it asymptotically approaches the given value.

5-4. ALLOWABLE SOIL BEARING PRESSURE

Equations (5-1) through (5-4) may be used to compute the ultimate bearing capacity of the soil at any predetermined depth below grade. This bearing capacity is used in combination with certain other factors to establish the allowable bearing pressure for which the footing may be designed. This pressure, symbolized by (q_a), is given by the following formula:

$$q_a = \frac{q_d - \gamma D_f}{SF} - \text{psf} \tag{5-5}$$

In which:

q_a = the allowable soil bearing pressure for which the footing may be designed based on the area of contact between the soil and the footing (psf)
q_d = the ultimate bearing capacity as determined from Equations (5-1) through (5-4) (psf)
γD_f = the weight of the soil above the base of the footing (psf)
SF = the safety factor

Note that the ultimate bearing capacity is reduced by the weight of earth above the base of the footing before the safety factor is applied.

A safety factor of 3 against footing failure by breaking into the soil is generally accepted by the construction industry. Some codes permit less, but remember—the architects and engineers are responsible for their work, not the writer of the code.

Simplified Equations for Footings on Cohesive Soil

The equations from which the allowable soil bearing pressure is computed for a footing bearing on pure clay can be simplified as indicated herein.

When computing the value of q_a for a continuous footing, Equation (5-5) can be written in the following form by incorporating Equation (5-1) into the numerator:

$$q_a = \frac{1.0\ cN_c + \gamma\ D_f N_q + 0.5\ B\gamma N_\gamma - \gamma D_f}{SF}$$

And noting that for a pure clay:

$$\phi = 0, \qquad N_c = 5.14, \qquad N_q = 1.0, \qquad N_\gamma = 0, \qquad c = 0.5\ q_u$$

Formula (5-5) then simplifies to:

$$q_a = \frac{0.5\ q_u\ 5.14}{3} = 0.857\ q_u$$

And finally:

$q_a = 0.86\ q_u$	for a continuous footing	(5-6)
$q_a = 1.02\ q_u$	for a square footing	(5-7)
$q_a = 1.02\ q_u$	for a round footing	(5-8)
$q_a = 0.86\ q_u\ a_l$	for a rectangular footing	(5-9)

5-5. PRESSURE BULBS

All footings impart pressure to the soil and the intensity of this pressure varies throughout the soil mass. The pressure is greatest directly beneath the center of the footing, and then spreads out into the soil both laterally and vertically in ever diminishing intensities. This is called the pressure bulb effect. A typical pressure bulb is illustrated in Figure 5-3.

It is evident that the shape of the pressure bulb will vary with the shape of the footing. Continuous footings produce a pressure bulb that is essentially linear

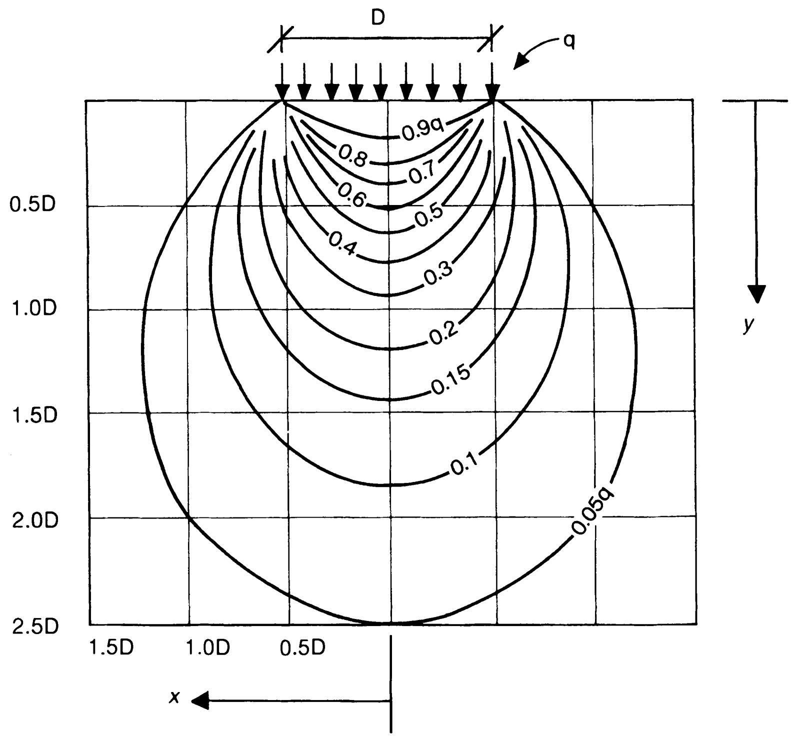


FIGURE 5-3. A typical pressure bulb, as produced by a circular footing. [Ref. 19]

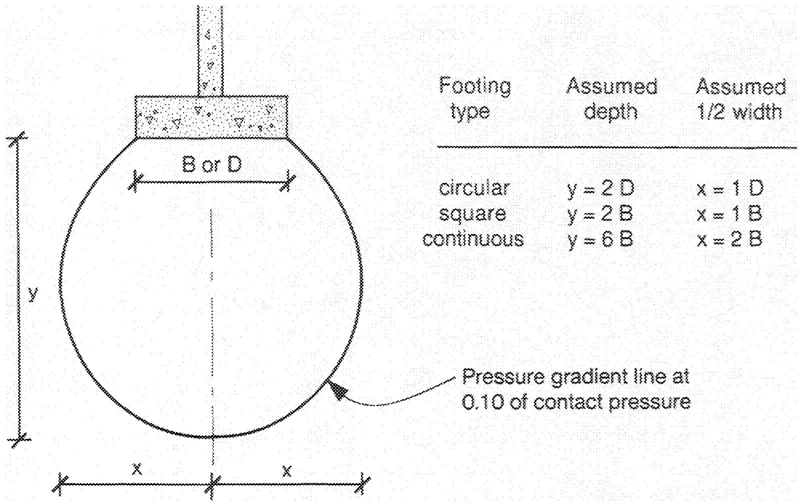


FIGURE 5-4. The depth and width at which the ten percent gradient line of the various pressure bulbs may be assumed to act. The maximum width of the bulb may be assumed to occur at mid-depth.

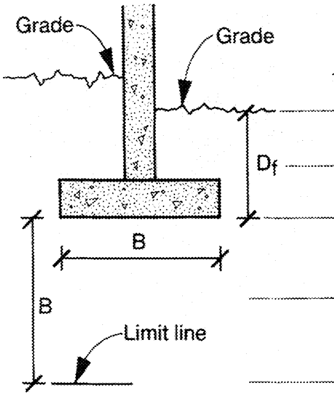
and approximates the shape of a continuous trench. The pressure bulb under an isolated footing spreads in all directions and approximates the shape of a bulb. Figure 5-3 illustrates the pressure bulb of a circular footing. The bulb of a square footing is essentially the same.

The pressure induced into a soil mass theoretically extends infinitely. It is recognized, of course, that this is not true. There comes a point where the inertia of the mass will no longer yield and at this point, for all intents and purposes, the pressure ceases.

For practical purposes, bearing pressure is usually of little consequence in the regions of soil extending beyond the 10% gradient line of the pressure bulb. The assumed extent of this gradient, in terms of width and depth, is given in Figure 5-4.

Footing Overlap

An examination of Figure 5-4 will show that the pressure bulbs of closely spaced footings will overlap whenever the center to center distance between them is less than the sum of their widths. This is cause for concern both in bearing and settlement considerations. A frequently used solution to this problem is to combine the two footings into one, as illustrated in Figure 7-6. This is discussed in Article 7-5.



Water level	Use in the $\gamma D_f N_q$ term	Use in the $B \gamma N_\gamma$ term
at 1	submerged weight	submerged weight
in 2	weighted average	submerged weight
at 3	normal weight	submerged weight
in 4	normal weight	weighted average
at 5	normal weight	normal weight

FIGURE 5-5. The reduction in unit weight of a soil due to ground water.

5-6. EFFECT OF GROUND WATER ON ULTIMATE BEARING CAPACITY

Ground water reduces the ultimate bearing capacity of all soils below the water table. The reason for this is the apparent reduction in soil density due to the effect of buoyancy. Whether the presence of ground water must be considered in computing the ultimate bearing capacity of any particular footing depends upon the elevation of the water table with respect to the footing. Such an adverse condition will occur whenever the water table is above an arbitrarily chosen limit line located a distance beneath the footing equal to the width of the footing. This condition and the modifications required by it are indicated in Figure 5-5.

It must be noted that ground water may fluctuate seasonally or may be susceptible to short term variation due to flooding or drought. It is the responsibility of the designer to take these matters into consideration.

When making calculations, remember that the unit weight of the submerged soil must be reduced by the unit weight of water:

$$\gamma_{sub} = (\gamma - 62.4) - \text{pcf}$$

5-7. RECOMMENDATIONS

General

As previously noted, a footing must be designed so that it will not break into the soil, nor exhibit excessive settlement. Footing design usually proceeds along the following lines:

1. The allowable soil bearing pressure q_a is computed using Formula (5-5). The required footing area is then computed by dividing the load imposed on the footing by q_a.

2. Several representative footings are then checked for settlement using the procedures outlined in Chapter 6. If these footings exhibit excessive settlement, the value of q_a must be reduced to provide a larger bearing area and, consequently, a smaller settlement. Subsequent design can usually be based on this reduced soil bearing pressure without the need to make continuing calculations on settlement.

Settlement calculations are presented in the following articles:

1. For footings on granular soil—Article 6-2.
2. For footings on cohesive soil—Article 6-5.

The procedure by which settlement is calculated for a mixed grained soil is a matter of engineering judgement. A mixed grained soil by definition includes both coarse and fine grained soil fractions. However, the characteristics and general behavior of any mixed grained soil will relate more to one of those fractions than to the other. In most instances the dominant fraction can readily be identified by visual examination and a physical manipulation of the soil. The dominant characteristics of each kind of soil are as follows:

1. Pure sand exhibits no cohesion and cannot be molded at all, nor can it be tested for unconfined compression strength.
2. Pure clay is inherently a cohesive material and can readily be molded. It also will exhibit a measurable unconfined compression strength.

There are a series of visual, hands-on tests that can be used to clearly identify the dominant fraction. These are described in Article 3-3. Once the dominant fraction of the soil been determined, settlement should be computed in accordance with the procedures outlined for that fraction.

Minimum Soil Pressure

It is the opinion of the author that soils having a computed allowable bearing pressure of one tsf or less, should rarely be used for the support of a building foundation, and that allowable pressures of less than 1½ tsf should be highly suspect. These low pressures invite problems with local shear and settlement.

Alternative solutions include removal of the offending soil, compaction, a mat foundation or the installation of deep foundations, such as piles or piers.

Local Shear

Footings founded on loose sand or soft or sensitive clay may fail at a lesser bearing pressure than that indicated by the equations. This condition is known as local shear. It has been suggested by Terzaghi and others that the numerical values of cohesion and of the angle of internal friction should be reduced by one-third in order to compensate for the possibility of footing failure due to local shear. This reduction will have the effect of substantially reducing the bearing capacity factors and the resultant soil bearing capacity. The problem facing the designer is to know under what circumstances he must consider local shear. This is a perplexing problem because there appears to be no certain line or transition which separates local shear from general shear. The terms loose sand, soft or sensitive clay, are approximations at best. It has been the experience of the author that the problems inherent with local shear can be avoided by never using any soil for bearing when the computed allowable bearing pressure is one ton per square foot or less.

As suggested reading, there is a very good discussion on the subject of local shear starting on page 207 of Lambe & Whitman's Soil Mechanics, SI Version, copyright 1969 by John Wiley & Sons, Inc.

Support Options

When the subgrade consists of satisfactory bearing material except for a relatively thin layer at the surface, this thin layer of unacceptable material should be removed and replaced with compacted borrow fill. In those instances when the material was unacceptable only because of density, it may be possible to densify this layer by compaction.

When the soil at the desired bearing elevation is relatively weak and is considered to be unacceptable for the support of individual footings, a mat or raft foundation may offer a reasonable alternative. This kind of foundation distributes the loads of many columns over a large area, considerably reducing the imposed soil pressure. A mat or raft foundation, however, will produce a much wider and much deeper pressure bulb which may have an adverse effect on settlement.

When the subgrade below the normal footing elevation is simply inadequate, and when this condition extends for some depth, the only alternative is to use a deep foundation that incorporates piles or piers. Such a foundation will safely transfer the loads to an acceptable bearing strata.

Minimum Footing Width

It has been the experience of the author that from the practical standpoint of digging a hole, maintaining the banks and placing reinforcement and concrete, the minimum width of any footing should be set as follows:

1. Two feet for the width of a continuous footing
2. Three feet for the sides of a square footing
3. Three feet for the smaller side of a rectangular footing

5-8. PRESUMPTIVE BEARING PRESSURE

Many building codes include a table of presumptive bearing pressures. These pressures are based on years of experience in the particular locality of which the code has jurisdiction. The use of these values is usually restricted by code to

TABLE 5-3. Presumptive Bearing Pressures

Soil Description	Bearing Pressure (tsf)
Very soft or soft clays	Not acceptable
Medium clay	1.5
Stiff clay	2
Very stiff clay	3
Hard clay	4–5
Very loose or loose sand	Not acceptable
Medium sand	1–3
Dense sand	3–4
Very dense sand	4–5
Gravel and gravel–sand mixtures	4–6
Compact silt or silty sand	1.5
Compact sandy clay	2
Compact silty clay	1.5
Organic soil	Not acceptable
Hardpan	10
Soft rock	8
Medium, sound rock	15
Hard rock	20–40
Massive, solid bedrock	100–200

projects of limited size, and they are not to be used for major work without substantiation by some kind of testing.

Presumptive bearing values as established by code have certain inherent disadvantages. Terminology, for example, can vary dramatically from one locale to another. In addition, these values usually do not consider variation of soil with depth, size of footing, ground water or permissible settlement. The only way in which to establish safe and cost effective bearing values is to conduct a testing procedure tailor-made for the particular project.

Presumptive bearing pressures do serve a useful purpose in that they provide a guideline of the kind of pressures that can generally be expected of a particular soil. Table 5-3 has been compiled by the author from different codes and from his own experience.

5-9. SAMPLE PROBLEMS

General Note

After making the calculations, the final answer is usually rounded off to a more convenient, more customary numerical value. Rounding off must be to a lower number. Soil pressure is usually specified in psf, ksf or tsf. The customary increments of roundoff are as follows:

500 psf, 0.5 ksf, 1 to 1½ tsf

Example 5-1

Given: A 2′ –6″ wide continuous footing bearing on mixed grained soil. The bottom of the footing is located 3′ –6″ below grade. Laboratory tests have determined:

$\phi = 28°$ $\gamma = 118$ pcf $q_u = 400$ psf $c = 0.5q_u = 200$ psf

Required: To calculate the allowable soil bearing pressure.

Bearing capacity factors from formulas: $N_c = 25.8$ $N_q = 14.7$ $N_\gamma = 11.2$

From Equation (5-1): $q_d = 1.0 \times 200 \times 25.8 + 118 \times 3.5 \times 14.7 + 0.5 \times 2.5 \times 118 \times 11.2 = 12{,}883$ psf

From Formula (5-5): $q_a = \dfrac{12883 - 118 \times 3.5}{3} = 4{,}156$ psf

use 4,000 psf

Example 5-2

Given: A 5′ –0″ square footing bearing on mixed grained soil. The bottom of the footing is located 6′ –0″ below grade. Laboratory tests have determined:

$\phi = 22°$ $\gamma = 115$ pcf $q_u = 1600$ psf $c = 0.5q_u = 800$ psf

Required: To calculate the allowable soil bearing pressure.

Bearing capacity factors from formulas: $N_c = 16.8$ $N_q = 7.8$ $N_\gamma = 4.1$

From Equation (5-2): $q_d = 1.2 \times 800 \times 16.8 + 115 \times 6.0 \times 7.8 + 0.4 \times 5.0 \times 115 \times 4.1 = 22{,}453$ psf

From Formula (5-5): $q_a = \dfrac{22453 - 115 \times 6.0}{3} = 7{,}254$ psf

use 7,000 psf

Example 5-3

Given: A 12′ –0″ diameter water tank, whose footing bears on mixed grained soil. The bottom of the footing is located 6′ –6″ below grade. Laboratory tests have determined:

$\phi = 18°$ $\gamma = 120$ pcf $q_u = 2400$ psf $c = 0.5q_u = 1200$ psf

Required: To calculate the allowable soil bearing pressure.

Bearing capacity factors from formulas: $N_c = 13.1$ $N_q = 5.3$ $N_\gamma = 2.0$

From Equation (5-3): $q_d = 1.2 \times 1200 \times 13.1 + 120 \times 6.5 \times 5.3 + 0.6 \times 6.0 \times 120 \times 2.0 = 22{,}862$ psf

From Formula (5-5): $q_a = \dfrac{23862 - 120 \times 6.5}{3} = 7{,}694$ psf

use 7.5 ksf

Example 5-4

Given: A 4′ –0″ × 12′ –0″ rectangular footing bearing on mixed grained soil. The bottom of the footing is located 4′ –6″ below grade. Laboratory tests have determined:

$\phi = 26°$ $\gamma = 116$ pcf $q_u = 800$ psf $c = 0.5q_u = 400$ psf

Required: To calculate the allowable soil bearing pressure.

Bearing capacity factors from formulas: $N_c = 22.3$ $N_q = 11.9$ $N_\gamma = 8.0$

From Table (5-1): $a_1 = 1.07$ $a_2 = 0.46$

From Equation (5-4): $q_d = 1.07 \times 400 \times 22.3 + 116 \times 4.5 \times 11.9 + 0.46 \times 4.0 \times 116 \times 8.0 = 17{,}463$ psf

From Formula (5-5): $q_a = \dfrac{17463 - 116 \times 4.5}{3} = 5{,}647$ psf

use 5.5 ksf

Example 5-5

Given: A 6′ –0″ square footing bearing on a grayish medium sand. The bottom of the footing is located 8′ –0″ below grade. Laboratory tests have determined:

$\phi = 33°$ $\gamma = 100$ pcf $q_u = 0$ $c = 0$

Required: To calculate the allowable soil bearing pressure.

Bearing capacity factors from formulas: $N_q = 26.1$ $N_\gamma = 26.2$

Note: In this example the value of N_c is not required because the soil has been specified as sand.

From Equation (5-2): $q_d = 100 \times 8.0 \times 26.1 + 0.4 \times 6.0 \times 100 \times 26.2$
$= 27{,}168$ psf

From Formula (5-5): $q_a = \dfrac{27168 - 100 \times 8.0}{3} = 8{,}789$ psf[a]

use 8.5 ksf

a. Settlement calculations for this footing will be made in Example 6-1. If the resultant settlement is considered excessive, the value of q_a must be reduced.

Example 5-6

Given: A 4′ –0″ × 8′ –0″ rectangular footing bearing on dense sand. The bottom of the footing is located 6′ –0″ below grade. Laboratory reports indicate:

$\phi = 36°$ $\gamma = 110$ pcf $q_u = 0$ $c = 0$

Required: To calculate the allowable soil bearing pressure.

From Table 5-1: $a_2 = 0.45$

Bearing capacity factors from formulas: $N_q = 37.8$ $N_\gamma = 44.5$

Note: In this example, as before, the value of N_c is not required.

From Equation (5-4): $q_d = 110 \times 6.0 \times 37.5$
$+ 0.45 \times 4.0 \times 110 \times 44.5 = 33{,}561$ psf

From Formula (5-5): $q_a = \dfrac{33561 - 110 \times 6.0}{3} = 10{,}967$ psf[a]

use 5 tsf

a. Settlement calculations for this footing will be made in Example 6-2. If the resultant settlement is considered excessive, the value of q_a must be reduced.

Example 5-7

Given: A 2′ –0″ wide continuous footing bearing on a stiff clay. The bottom of the footing is located 4′ –0″ below grade. Laboratory tests have determined:

$\phi = 0°$ $\gamma = 122$ pcf $q_u = 3000$ psf $c = 0.5q_u = 1500$ psf

Required: To calculate the allowable soil bearing pressure.

Bearing capacity factors from formulas: $N_c = 5.14$ $N_q = 1.0$ $N_\gamma = 0$

From Equation (5-1): $q_d = 1.0 \times 1500 \times 5.14 + 122 \times 4.0 \times 1.0$
$= 8{,}198$ psf

From Formula (5-5): $q_a = \dfrac{8198 - 122 \times 4.0}{3} = 2{,}570$ psf

Or by using the simplified method, from Formula (5-6):

$$q_a = 0.86 \times 3000 = 2{,}580 \text{ psf}$$

This allowable soil bearing pressure is very low and would not normally be considered as an acceptable design criterion for spread footings. A mat foundation may be an acceptable alternative. The weight of the mat, however, may reduce the available pressure to an unacceptable level. Situations like this are usually soled with deep foundations, such as piles or piers.

Example 5-8

Given: A 3′ –0″ × 6′ –0″ rectangular footing bearing on a very stiff clay. The bottom of the footing is located 7′ –0″ below grade. Laboratory tests have determined:

$\phi = 0°$ $\gamma = 125$ pcf $q_u = 6000$ psf $c = 0.5q_u = 3000$ psf

Required: To calculate the allowable soil bearing pressure.

Bearing capacity factors from formulas: $N_c = 5.14$ $N_q = 1.0$ $N_\gamma = 0$

From Table 5-1: $a_2 = 1.12$

From Equation (5-4): $q_d = 1.12 \times 3000 \times 5.14 + 125 \times 7.0 \times 1.0$
$= 18{,}145$ psf

From Formula (5-5): $q_a = \dfrac{18145 - 125 \times 7.0}{3} = 5{,}756$ psf

Or by using the simplified method, from Formula (5-9):

$$q_a = 0.86 \times 6000 \times 1.12 = 5{,}779 \text{ psf}$$

use 2½ tsf

Example 5-9

Given: The 12′ –0″ tank footing of Example 5-3.

Required: To calculate the allowable soil bearing pressure, as assuming that ground water is 2′ –0″ below grade.

From Figure 5-5, the water table is located in level 2. Therefore, when applying Equation (5-3), use the weighted average in the second term, and use the submerged weight in the third term.

The submerged weight = 120.0 – 62.4 = 57.6 pcf

$$\text{The weighted average } = \frac{120.0 \times 2.0 + 57.6 \times 4.5}{6.5} = 76.8 \text{ pcf}$$

From Equation (5-3): $q_d = 1.2 \times 1200 \times 13.1 + 76.8 \times 6.5 \times 5.3$
$+ 0.6 \times 6.0 \times 57.6 \times 2.0 = 21{,}924$ psf

$$\text{and } q_a = \frac{21924 - 76.8 \times 6.5}{3} = 7{,}141 \text{ psf}$$

use 7.0 ksf

In this particular example, the reduction in soil capacity due to the existence of ground water was approximately 7%.

Example 5-10

Given: A 3′ – 0″ wide continuous footing located 3′ – 6″ below grade. The soil is found to be mixed grained and is described as a very loose soil. Laboratory tests have found:

$\phi = 22.5°$ $\gamma = 105$ pcf $q_u = 1200$ psf $c = 0.5q_u = 600$ psf

Required: To calculate the allowable soil bearing pressure.

When the supporting soil is described as very loose or very soft, footing failure may occur because of local shear rather than general shear. Terzaghi and others have recommended that for this condition the angle of internal friction should be reduced by one-third before determining the bearing capacity factors. The numerical value of the cohesion should likewise be reduced by one-third.

Therefore: the effective angle of internal friction = 22.5 × ⅔ = 15°
and the effective cohesion = 600 × ⅔ = 400 psf

Bearing capacity factors from formulas: $N_c = 11.0$ $N_q = 3.9$ $N_\gamma = 1.1$

From Equation (5-1): $q_d = 400 \times 11.0 + 105 \times 3.5$ z 3.9
$+ 0.5 \times 3.0 \times 195 \times 1.1 = 6{,}006$ psf

From Formula (5-5): $q_a = \dfrac{6006 - 105 \times 3.5}{3} = 1{,}880$ psf

It is the opinion of the author that the allowable soil bearing pressure, as determined in this example, is so low that the premise of using spread footings for this condition is unacceptable. The reader should refer to a discussion of minimum soil bearing pressure and local shear in Article 5-7.

The possibility of a mat foundation was explored in Example 5-7, where the soil bearing pressure also was low. In this particular problem, the weight of the mat would reduce the available pressure to the point of impractibility. The use of piles or piers appears to be the only viable solution.

6

Settlement Analysis

6-1. INTRODUCTION

General Considerations

Settlement is the term used to describe the action by which a footing pushes into the ground in response to the load to which it is subjected. The amount of settlement is a function of the size of the footing, the load to which the footing is subjected, and the characteristics of the soil directly beneath and, for some distance, below and beyond the footing. There are procedures by which the amount of settlement can be approximated for any given condition of footing size, loading, and soil characteristics. The experience gained by observation of the time-related performance of countless buildings has given the architect and engineer insight as to how much settlement can safely be permitted under a given set of circumstances. Settlement can never be eliminated, unless bearing on solid rock. It can be controlled, however, by selecting the type of foundation best suited to the type of soil found at the site, and then by establishing an allowable soil bearing pressure as a function of allowable settlement.

The consequences of using the wrong type of foundation on the wrong type of soil can be disastrous. A famous example of such a disaster is the Leaning Tower of Pisa, construction of which was started in 1174. This tower, as illustrated in Figure 6-1, is 179 feet high with an offset that presently measures 17 feet. Although extensive repairs have been made in an effort to stabilize the soil upon which the foundations were built, the amount of offset continues to increase at a rate of approximately ¼″ per year. Engineers have indicated that the tower must soon be torn down.

In Article 5-5 the concept of the pressure bulb was introduced. It was noted that the pressure induced into the soil by an applied load is usually of little

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

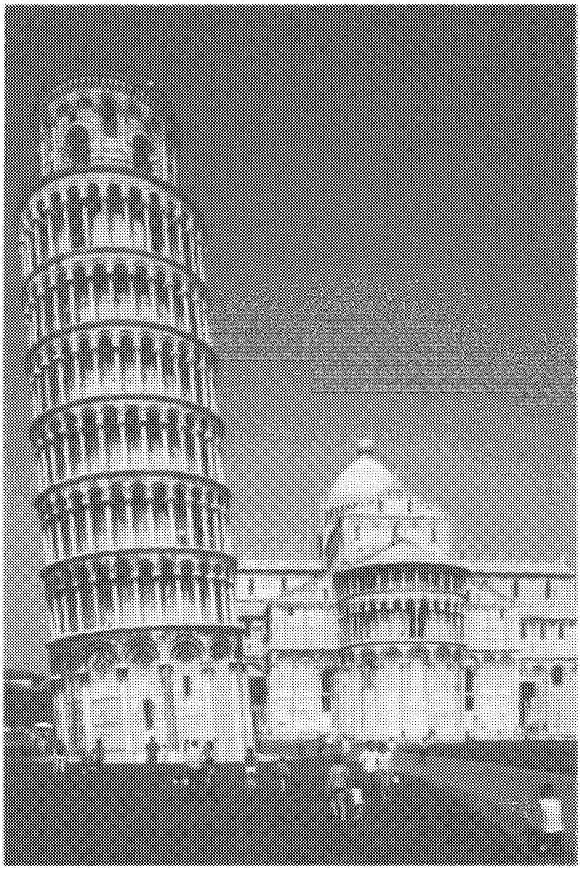


FIGURE 6-1. The Leaning Tower of Pisa. An example of differential settlement.

consequence in regions of soil beyond the ten percent gradient line of the pressure bulb. The same is true of settlement. It must be noted, however, that soil within this pressure bulb is usually not homogeneous but may be made up of layers of different soils having different characteristics. The settlement attributible to each of these layers must be computed individually and then added together to determine the overall settlement. The procedure by which these calculations are made differs between sand and clay and will be described in subsequent paragraphs.

Effect of Pressure Bulb Sizes

The size of the pressure bulb and the depth to the one-tenth point varies with the size and type of footing, as indicated in Figure 5-4. When two footings are of different size but have the same contact pressure, the larger footing will have a correspondingly larger pressure bulb. This can present a problem in settlement as illustrated in Figure 6-2.

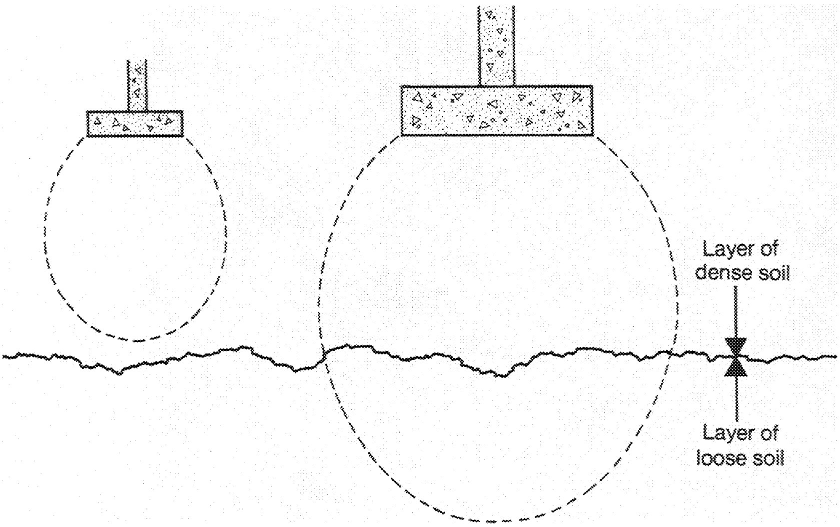


FIGURE 6-2. The size of the pressure bulb may have considerable effect on footing settlement.

The pressure bulb of the smaller footing is contained solely within the upper layer of soil. Settlement for this footing, therefore, depends solely on the response of that soil to load. The pressure bulb of the larger footing extends down through the upper layer into the layer beneath. Settlement of this footing, therefore, depends on the sum of the responses of each individual layer.

Adjustment to Allowable Bearing Pressure

Settlement due to total load usually has a specified limit. If the computed settlement exceeds this limit, the values of q_a and P must be reduced proportionately.

Reasons for Settlement on Sand

Sand beds usually contain particles having a variety of sizes and shapes. In certain instances, as when the sand is below the water table, the smaller particles may settle out, and the larger particles must then arch over the voids. Such an arrangement of particles is referred to as a honeycomb structure. Although the particles in a honeycomb structure have sufficient strength to arch over the relatively large voids thus produced, as well as to carry the weight of the overlying soil, they may have insufficient contact between them to carry additional load. An example of such a particle arrangement is illustrated in Figure 6-3. An examination of this arrangement of particles shows that, although there is particle to particle contact throughout the mass, the arrangement is not stable due to the high volume of voids. When subjected to a superimposed load, the particles will shift and rearrange themselves until a relatively solid mass is formed. This action

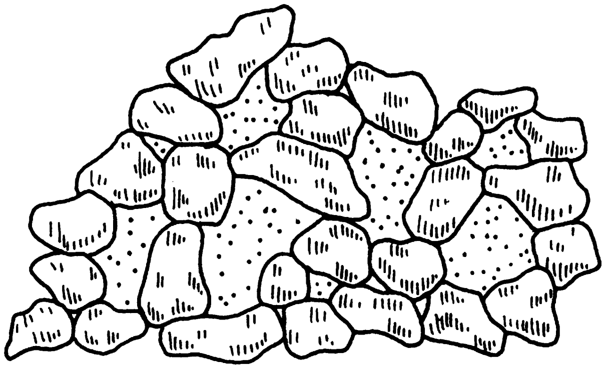


FIGURE 6-3. Honeycomb structure in a granular soil before intergranular slippage occurs. [Ref. 10]

is called intergranular slippage. The extent of this slippage and, therefore, the amount of settlement depends upon the variation of particle size and in their shape and distribution within the mass. If all sands were round in shape and uniform in size, such a rearrangement would not occur since each particle would already be in stable contact with the adjacent particles. Such a condition would be analogous to a box of uniformly sized marbles. Sand, however, is not uniform. It comes in all shapes and sizes.

Figure 6-4 illustrates the effect of intergranular slippage on the particle arrangement previously indicated in Figure 6-3. In this instance the particles were acted upon by an external force. This could have been caused by a superimposed load, by compaction or by vibration. But in either case, the particles were shifted and rearranged into a relatively solid mass quite capable of supporting major loads without appreciable settlement.

Lateral yielding of the soil is a second reason why settlement will occur under load. The pressure exerted by a footing dissipates into the ground in all directions as indicated by the pressure bulb concept. If the soil which encloses the pressure

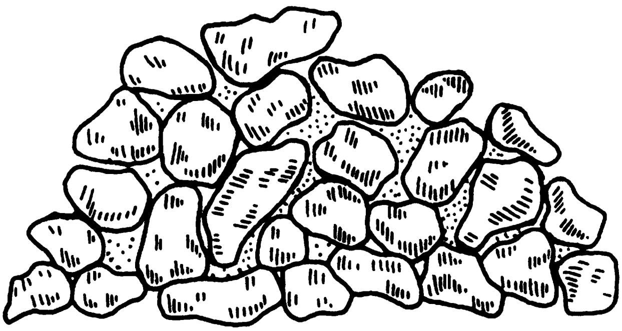


FIGURE 6-4. The effect of compaction on the honeycomb structure of Figure 6-3.

bulb is relatively loose, it will yield when subjected to lateral pressure. As the soil yields, it densifies. The soil will continue to yield until there has been sufficient densification to prevent further yielding.

A volume of soil responds to loads much like a balloon filled with air. When subjected to a downward load the balloon and the soil will compress vertically and push out laterally. The amount of lateral movement and, therefore, the amount of settlement is dependent on the lateral resistance offered by the enclosing soil mass. Loose soils, of course, are much more susceptible to large and quite possibly unacceptable settlements. The standard penetration test, as described in another section, provides immediate insight as to the density of any deposit of soil. This is because it has been determined that the number of blows recorded as the sampler spoon is advanced into the soil is, to a certain extent, a measurement of the soil density.

No individual particle of soil is incompressible. Therefore, each particle will compress under load and will theoretically contribute to settlement. This is called elastic deformation. The settlement caused by this deformation is insignificant compared to that of intergranular slippage and lateral yielding and is normally ignored.

Reasons for Settlement of Clay

The settlement of a footing on clay, for all intents and purposes, is solely a function of reduction in void ratio caused by the loss of air and water from the voids. This action is analogous to that of squeezing a sponge whose voids are filled with air and water. It may be remembered that clay contains two different kinds of water: free water (frequently called pore water) and intergranular absorbed water. As pressure is exerted on the clay layer, the entrapped air is expelled rather quickly. There is, however, only a small amount of air in the voids, as they are filled mostly with water. The applied pressure forces the free water to begin to migrate out of the loaded area. This migration and the settlement which it causes may be slow or quick depending on the permeability of the soil. In most instances the migration will be very slow. Slower yet will be the release and subsequent migration of the intergranular absorbed water. The interaction between clay and water is discussed in Article 13-2.

Rate of Settlement

Settlement is a time related phenomenon, and may occur progressively over an extended period of time. The rate of settlement depends on the type of material which makes up the underlying ground. Sand compacts quickly when subjected to load while clays compact much more slowly. Since the majority of the load of a building is usually its own dead weight, it follows that buildings built on sand experience most of their settlement during construction. Buildings built on clay, however, experience most of their settlement while in service. A general

TABLE 6-1. **Rate of Settlement[a]—Sand and Clay**

	During Construction (%)	During Service (%)
Footings on sand	90	10
Footings on clay	10	90

[a] Percentages are for general guidelines only.

rule of thumb regarding the percentage of total settlement that a building can expect to experience over a period of time is given in Table 6-1.

Permissible Settlement

No building can be designed to eliminate settlement completely, unless its foundations bear directly on solid rock. Settlement will occur in all other buildings, regardless of the type of foundation and of the kind of soil upon which they bear. A building can experience two different kinds of settlement:

1. Settlement may occur uniformly distributed over the entire area of the building. This kind of settlement will not cause damage to individual parts of the building, but may cause misalignment at points of egress and may lead to serious breakdowns in mechanical services, such as in sewer or gas lines.
2. Settlement may occur differentially between different parts of the building. This kind of settlement may cause serious damage to individual parts of the building, and can also lead to progressive failure and ultimate collapse of the structural system.

In order to avoid the adverse effects of excessive settlement a limit must be placed on the amount of settlement that can be tolerated. The building must then be designed architecturally, structurally and mechanically to work within this limit. Unless more stringent limits are required by the governing building code or by the sensitivity of any particular building elements to settlement, it is considered reasonable by most authorities to limit the projected total settlement due to dead load and live load as follows:

1. The maximum settlement of individual footings should generally be limited to 1½ inch, although this limit may be reduced to 1 inch by code or by the judgement of the engineer.
2. The differential settlement between any two adjacent footings should be limited to 1/360th of the horizontal distance between them, measured in inches.

It should be noted that the superimposed dead load for which a footing may be responsible can be determined with considerable accuracy. Live load, however, is a completely different matter and footings are rarely subjected to their full basic live load. Living areas in hotels and apartment houses and patient rooms in hospitals, for example, usually carry a code specified live load of 40 psf. Realistic live loads in these areas, however, is more likely to be 10 to 15 psf. This is why building codes permit a reduction of live load under certain conditions of occupancy and area of responsibility. Live load, it is seen, is a local thing. As area is increased the resulting unit live load is decreased. It is for this reason that some columns and footings may have their design live load reduced by as much as 60%.

It should also be noted that, as noted in Table 6-1, 90% of the settlement of a foundation bearing on sand will occur during construction. Much of this settlement will occur, therefore, before the finish materials of the building are attached. This has the effect of lessening the impact of settlement on coarse grained, sandy soils.

6-2. SETTLEMENT CALCULATIONS—FOOTINGS ON SAND

It was previously noted that the settlement of a footing on sand is due primarily to the combined effects of intergranular slippage and lateral yielding. These effects occur quickly as the load is applied to the soil. After the particles of soil have shifted into a more stabilized arrangement and after the soil mass has been further densified through the process of lateral yielding, additional load will cause little additional settlement. In the majority of cases the dead weight of the building is much heavier than the actual live load (as opposed to the design live load) to which the building will be subjected. Most of the weight will therefore be added during construction. It is for these reasons that up to 90% of the total settlement of a building can be expected to occur during construction.

Different methods have been advanced as to the computation of settlement for a footing bearing on sand. One method is based on making a load-settlement test in the field. The problem with this method is the considerable difference in size between the pressure bulb of the comparatively small test bearing plate and that of the actual footing. It is the opinion of the author that this difference provides serious challenge as to the validity of the test. Laboratory analysis of samples of sand do little to indicate the settlement characteristics of a natural deposit of sand. Other methods of settlement analysis are therefore empirical and are primarily based on the density of the soil: the denser the soil, the lesser the settlement.

It has been established over a period of years that there is a general correlation between the relative density, density, blow count and angle of internal friction of a coarse grained soil. These correlations are given in Tables 2-2 and 3-2 and Figure 3-10. For the convenience of the reader this information has been combined in Table 6-2.

TABLE 6-2. Summary of Relative Density, Unit Weight, Blow Count, and Angle of Internal Friction For Coarse Grained Soils[a]

Description	D_r	γ	N	ϕ
Loose	<35	<90	4–10	<30
Medium	35–65	90–110	10–30	30–36
Dense	65–85	110–130	30–50	36–41
Very dense	>85	>130	>50	>41

[a] Compiled from Tables 2-2 and 3-2, and Figure 3-10.

In those instances where this correlation appears to break down, the blow count is the one that should usually be suspect. The accuracy of the blow count can be affected by a variety of things, including:

1. Pebbles or small pieces of stone can impede the driving of the casing or the sampler spoon.
2. Water from any number of sources may accelerate the driving by lubricating the casing or the sampler spoon.

When the blow count is truly suspect, its value should be corrected so as to agree more closely with the angle of internal friction of the sample taken at that depth.

There are two recommended procedures by which settlement can properly be considered in the design of a building foundation.

1. Calculate the anticipated settlement by using the empirical formula included in this article. If the calculated settlement exceeds the allowable, the allowable bearing pressure must be reduced, thereby increasing the size of the footing.
2. The use of design charts such as those developed by Peck, Hanson, and Thornburn [Ref. 16, p. 309]. This type of chart provides a direct design based on a combination of bearing pressure and permissible settlement.

The calculations required to determine settlement in accordance with the first procedure are as follows:

1. Compute the allowable bearing pressure q_a, using the method outlined in Article 5-4.
2. Compute the footing capacity P, by multiplying the footing area by q_a.
3. Compute the anticipated settlement, using the following empirical formula:

$$\Delta = \frac{4 \times P}{N \times (B + 1)^2} \text{ [Ref. 14]} \qquad (6\text{-}1)$$

in which

Δ = the anticipated settlement (inches)
P = the design load on the footing (kips)
N = the lowest corrected blow count occurring in a depth measured from the base of the footing to a point below the footing equal to its width B
B = the width of a continuous footing, the side of a square footing, the least width of a rectangular footing, or the diameter of a round footing (feet)

Values of N, as recorded during the standard penetration test, are subject to modification due to (a) submergence by ground water, or (b) release of overburden. The coefficients C_W and C_N as described in Article 3-7, shall be used for this purpose.

6-3. THE THEORY OF SETTLEMENT—FOOTINGS ON CLAY

General

Settlement of footings on clay is directly attributable to a reduction in void ratio, which is caused by a reduction in volume, which is caused by an increase in pressure. The amount of settlement depends primarily on:

1. The intensity of the applied load
2. The load responsive characteristics of the particular clay
3. The history of loading to which the clay has been subjected throughout an extended period of time

New Terms

A discussion of the past history of loading requires the introduction of the following new terms:

Overburden: Overburden is simply the dead weight of any earth occurring above any horizontal plane of reference within the soil mass.

Normally loaded soils: When a soil is referred to as being normally loaded, it means that the soil has never, in all of its history, been loaded with a greater overburden than the one existing at present. A normally loaded soil is sometimes referred to as normally consolidated.

Preloaded soils: A preloaded soil is one that at some time in its history was loaded with a greater overburden than the one existing at present. This condition is usually the result of glacial ice that covered much of the land masses many years ago. The weight of this ice compressed the soil and densified it. The condition existing at the present time is the result of the melting of the ice.

TABLE 6-3. Differences Between Normally Loaded and Preloaded Clays

Property	Normally loaded	Preloaded
Compressibility	High	Low
Bearing capacity	Low–medium	Medium–high
Water content[a]	Close to LL^b	Much less than LL^b

[a]This comparison can usually identify whether a soil is normally loaded or preloaded.
[b]For information on liquid limit, refer to Article 13-6.

Although it is true that the soil volume has partially rebounded, there is still a certain amount of residual compression. A preloaded soil is sometimes referred to as precompressed, preconsolidated, or overconsolidated.

Normally loaded and preloaded clays exhibit several very different characteristics, as itemized in Table 6-3.

Relationship Between Settlement, Void Ratio, Volume, and Pressure

The relationship of these four items is illustrated in Figure 6-5. It can be seen that settlement is basically caused by an increase in pressure. The medium through which it works is the reduction in volume and void ratio.

Settlement is linear rather than volumetric. In order for the concept of settlement to be truly linear, as described herein, it must be assumed that the soil under discussion is restrained against lateral deformation.

In Figure 6-5 and in the calculations that follow, subscript (o) refers to the original geometry of the soil; subscript (f) refers to the final geometry of the soil after it has been subjected to an increase in loading.

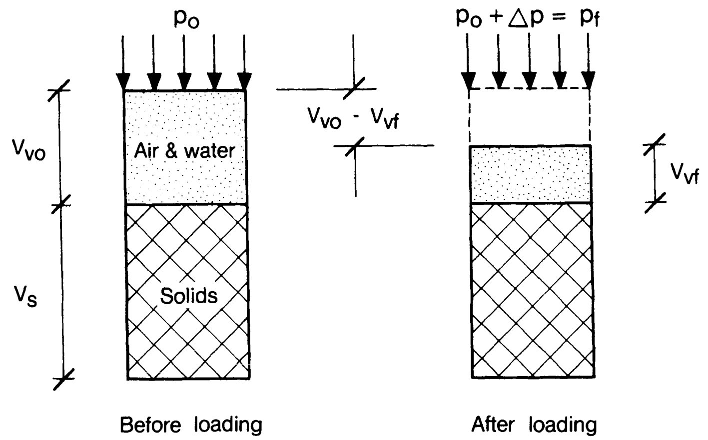


FIGURE 6-5. Settlement as a function of reduction in void ratio.

The void ratios of the soil before and after being subjected to an increase in load are, by definition:

$$e_o = \frac{V_{vo}}{V_s} \qquad \text{and} \qquad e_f = \frac{V_{vf}}{V_s}$$

When the volume of solids is taken as unity the decrease in volume can be expressed as a corresponding decrease in void ratio:

$$V_{vo} - V_{vf} = e_o - e_f$$

It is this decrease in void ratio from which calculations of settlement are based. Because this decrease in void ratio is caused by an increase in pressure it is necessary to determine the relationship between them. This can be done in the laboratory by performing a consolidation test on an undisturbed sample of the clay soil.

6-4. CONSOLIDATION TEST

Consolidation is a term used by architects and engineers to describe the phenomenon by which clay and soils whose characteristics are predominantly those of clay will densify when subjected to an increase in pressure. Therefore, consolidation is always accompanied by a reduction in volume and, correspondingly, a reduction in void ratio.

A consolidation test is a specialized test performed in the laboratory on an undisturbed sample taken from the actual clay layer being analyzed. The test must be conducted in accordance with the following ASTM Standard:

> D-2435: Standard Test Method for One-Dimensional Consolidation Properties of Soils

The size of sample used for this test is optional with the laboratory provided that it meets the requirements of the ASTM Standard. Several tests with which the author was familiar were made with a one inch high sample having a diameter of 2½″.

It has been previously noted that settlement is linear. The apparatus used in this test provides for the continued lateral restraint of the sample. This is done so that the deformation experienced by the sample will be purely linear.

The primary purpose of this test is to determine the variation in void ratio as a function of pressure. This will lead directly to computations of settlement. The secondary purpose of the test is to determine the continuance of deformation over time. This is called the time rate of consolidation. This test will give insight as to how quickly or how slowly settlement will occur.

This test is performed by applying an axial load to the sample. This causes the sample to undergo a gradual reduction in height due to the time release of air and water from the voids. Remember, the sample is restrained against lateral movement so all deformation is linear. During the test the reduction in height is recorded at various intervals. The test load is maintained at a constant level until the reduction in height has, for all intents and purposes, ceased. The test is then repeated several times, but each time with a greater load. The usual loads are those which induce an axial stress of 100, 200, 400, 800, 1600, etc., pounds per square foot to the sample being tested. The tests should be continued, with doubled loads, until the test load exceeds the load to which the clay deposit will be subjected by the weight of the structure it is expected to support. This series of tests is normally applied in sequence to a single sample. The gauge that records the deformation is then reset before the next load is applied to the sample. It should be noted that readings of total deformation will be cumulative.

Deformation as a Function of Time

The information obtained from the consolidation test is first used to develop a set of curves which express deformation as a function of time for different conditions of loading. Deformation, in this case, is numerically equal to the reduction in the height of the sample. One such set of curves is shown in Figure 6-6.

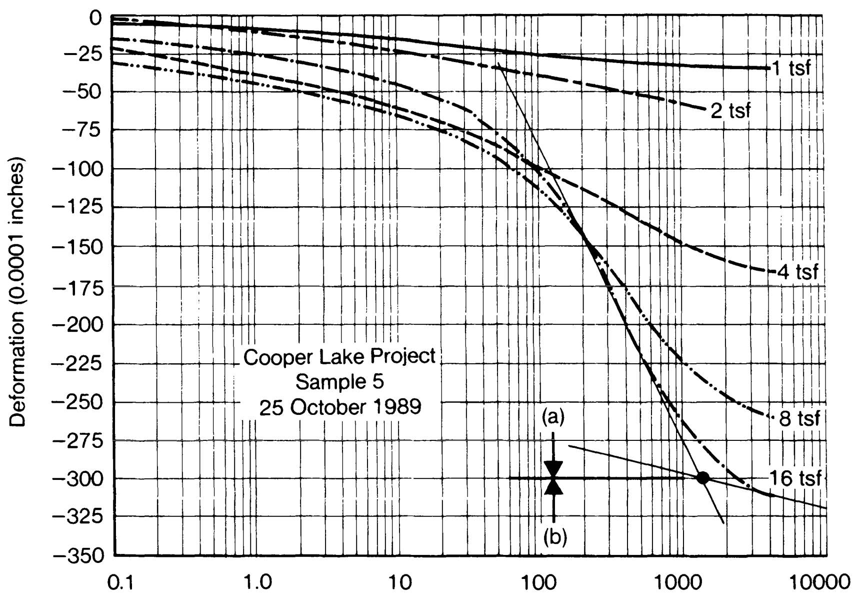


FIGURE 6-6. The results of a particular consolidation test in which deformation was determined as a function of time. (**a**) Range of primary consolidation. (**b**) Range of secondary compression. [Ref. 22]

Long term consolidation is the result of two different actions. The first is the squeezing of water out of the voids. This is called primary consolidation. The second is the readjustment of the soil grains within the soil mass. This is called secondary compression. Of the two, primary consolidation is by far the more critical when considering the settlement of a building. It is for this reason that ASTM D-2435 includes a procedure whereby the transition from primary consolidation to secondary compression can be identified in terms of deformation. This deformation is determined graphically as the intersection of two straight lines drawn on a particular time-deformation curve. The first line is drawn tangent to the curve where it assumes a relatively flat slope. The second line is drawn tangent to the steepest part of the curve. These lines and their point of intersection are shown, for the purpose of illustration, on the 16 tsf curve. The point of intersection of these two lines identifies the deformation corresponding to 100% consolidation. In Figure 6-6, this point occurs at a deformation of 0.0300 inches. Compression occurring subsequent to this deformation is secondary compression.

The *e*-log *p* Curve

As previously noted, the primary purpose of performing a consolidation test is to determine the variation in void ratio as a function of pressure. The deformation experienced by the sample during the loading cycle provides the necessary information whereby this variation can be determined.

The first computations relate to a cubic foot of earth. The dry unit weight and the specific gravity of the solids have been determined prior to the start of the test procedure. These values can be used to determine the volume of solids by rearranging Formula (2-7):

$$V_s = \frac{W_s}{62.4\ G_s}$$

The volume of voids is then determined:

$$V_v = 1 - V_s$$

The void ratio existing at the start of the test is:

$$e_o = \frac{V_{vo}}{V_s}$$

Subsequent measurements and computations taken during the test relate to the geometry of the sample, of which the diameter and initial height must be known.

Rather than deal with volumes, it is more convenient to convert the formula for void ratio to heights.

$$e = \frac{V_v}{V_s} = \frac{h_v}{h_s} = \frac{h - h_s}{h_s}$$

where h is the original height of the sample.

It is noted that the equivalent height of solids in the sample can be determined by multiplying the volume of solids in one cubic foot by the initial height of the sample.

After the time related reduction in sample height has ceased the resulting void ratios are computed for each of the test pressures and plotted on an e-log p curve, a example of which is shown in Figure 6-7. In the consolidation test illustrated in this example the test was extended to include readings taken as the pressure was unloaded. This was done in order to gain information as to the swelling characteristics of the soil as pressure is reduced.

It must be noted that the consolidation test as described herein is both costly and time consuming. A single operation may take up to 24 hours to complete, and an entire test may take several weeks. It is, however, the best method available from which critical building settlements may be anticipated with a reasonable assurance of accuracy.

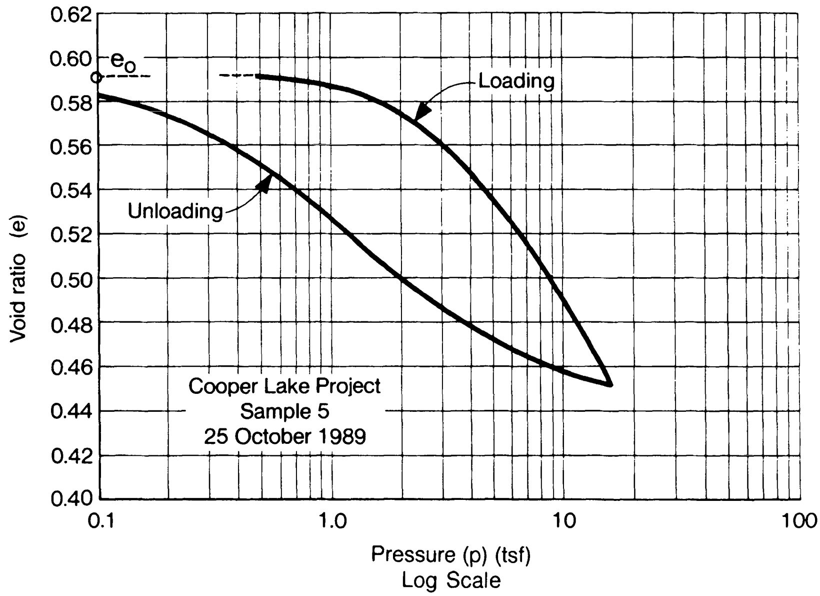


FIGURE 6-7. A continuation of the consolidation test of Figure 6-7, expressing the variation in void ratio as a function of pressure. This is commonly called an e-log p curve. [Ref. 22]

6-5. SETTLEMENT CALCULATIONS—FOOTINGS ON CLAY

Linear Strain and Settlement

By definition, linear strain is the ratio of the reduction in length to the original length. Therefore:

$$\text{Linear strain} = \frac{V_{vo} - V_{vf}}{V_s - V_{vo}} = \frac{e_o - e_f}{1 + e_o}$$

This transfer from volume to void ratio can be obtained by dividing the numerator and denominator by V_s.

If the linear strain is assumed to be relatively constant throughout a layer of clay, the total settlement in that layer will be the product of the linear strain times the height of the layer. The application of this principle leads to the following formula for settlement:

$$\Delta = \left[\frac{e_o - e_f}{1 + e_o}\right] H \tag{6-2}$$

The variables used in the application of Formula (6-2) may be defined as follows, for a given layer of soil:

- $\Delta =$ the anticipated settlement (inches)
- $H =$ the height of the layer (inches)
- $e_o =$ the void ratio of the layer prior to the start of construction—e_o corresponds to p_o
- $e_f =$ the void ratio of the layer after completion of construction—e_f corresponds to p_f—its numerical value is read off of the e-log p curve

In order to apply Formula (6-2) certain pressures are required, each of which is computed on a plane which occurs at the mid-depth of the layer of soil under consideration:

- $p_o =$ the overburden pressure existing on the plane prior to the start of construction—if ground water is above the plane being considered, the submerged weight must be used for all earth between the plane and the water table
- $\Delta p =$ the increase in pressure on the plane due to the superimposed weight of the new construction—this pressure may or may not, at the discretion of the designer, include all or part of the live load for which the building is designed
- $p_f =$ the resultant pressure on the plane after construction has been completed—p_f is numerically equal to $p_o + \Delta p$

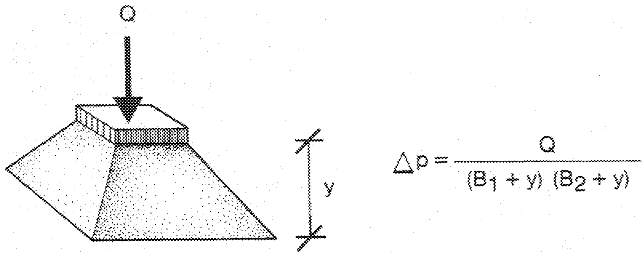


FIGURE 6-8. A conservative approximation for the increase in pressure Δp at any given depth below the footing.

> *Note: p* values are computed in (psf) and are then converted to (tsf) for use with the e-log p curve.

Remember that if the soil within the depth of the pressure bulb consists of different layers, each with their own settlement characteristics, settlements must be computed individually for each layer, after which they are added together.

Determination of Δp

Either of the three following procedures maybe used to compute the value of Δp:

(a) Using the pressure bulb theory as illustrated in Figure 5-3.
(b) Using the assumption that the distribution of pressure relates to the frustrum of a right pyramid, as illustrated in Figure 6-8.
(c) Using the theories presented in Appendix C.

The use of each of these procedures is demonstrated in Example 6-5.

The procedure described in item (b) is the one most frequently used by architects and engineers because of its relative simplicity and close agreement with other more academic considerations. In this procedure it is assumed that the superimposed load is distributed into the ground on a gradient whose shape is that of the frustum of a right pyramid. The sides of the pyramid are assumed to slope one unit horizontal for each two units vertical. The increase in pressure may be calculated directly by use of the formula given in the figure.

Settlement Calculation Procedure

Make a sketch showing a profile of the soil extending from grade down to the depth for which settlement calculations must be made. Include all available information for each layer of soil within the zone of interest. Certain information, such as blow count, water table and a description of the soil, can be obtained from the test borings. Other information, such as in-place density, dry density, moisture content and the specific gravity of the solids can be obtained from the laboratory analysis. Then:

1. Determine the allowable footing load using the procedures of Chapter 5.
2. Compute V_s, V_v, and e_o. Then perform a consolidation test, recording results in tabular form. Compute void ratios for all test loads, record on table and use these values to draw the e-log p curve.
3. Compute p_o, Δp, and p_f.
4. Read e_f from the e-log p curve. Then compute settlement for the layer of soil under consideration, using Formula (6-2). If settlement is excessive, reduce q_a and P accordingly.

Note: Repeat the above steps for all other layers of soil that occur within the 10% gradient zone. Obtain the total settlement by adding the individual settlements.

6-6. SAMPLE PROBLEMS

Example 6-1

Given: Refer to Example 5-5 in which a 6′ –0″ footing bears on grayish medium sand.

Required: To compute the anticipated settlement of this footing and to modify the previous allowable soil bearing pressure if the settlement exceeds 1 inch.

From Formula (6-1):
In order to use this formula, the blow count N must be determined. For the given value of $\phi = 33°$, select the companion value of $N = 20$ from Table 6-2. Then, using $q_a = 8{,}789$ psf from Example 5-5:

$$P = q_a \times \text{area} = 8.789 \times 6 \times 6 = 316 \text{ kips}$$

$$\text{and: } \Delta = \frac{4 \times 316}{20 \times (6 + 1)^2} = 1.29''$$

In order to limit the settlement to 1.00″, the original values for q_a and P must be reduced by dividing by 1.29. Therefore:

$$q_a = 6{,}813 \text{ psf} \qquad \text{and} \qquad P = 245 \text{ kips}$$

Example 6-2

Given: Refer to Example 5-6 in which a 4′ –0″ × 8′ –0″ rectangular footing bears on dense sand.

Required: To compute the anticipated settlement of this footing and to modify the previous allowable soil bearing pressure if the settlement exceeds 1 inch.

From Formula (6-1):
In order to use this formula, the blow count N must be determined. For the given value of $\phi = 36°$, select the companion value of $N = 30$ from Table 6-2. Then, using $q_a = 10{,}967$ psf from Example 5-6:

$$P = q_a \times \text{area} = 10.967 \times 4 \times 8 = 351 \text{ kips}$$

$$\text{and} \qquad \Delta = \frac{4 \times 351}{30 \times (4 + 1)^2} = 1.87''$$

In order to limit the settlement to 1.00″, the original values for q_a and P must be reduced by dividing 1.87. Therefore:

$$q_a = 5{,}864 \text{ psf} \qquad \text{and} \qquad P = 188 \text{ kips}$$

Note: A rectangular footing is penalized in the settlement formula because the least dimension occurs in the denominator. A square footing of equal area would be much more efficient.

Examples 6-3 through 6-6

These four problems deal with the analysis of the 8′–0″ square footing illustrated in Figure 6-9. This analysis will consider soil bearing pressure and settlement.

Given: Soil (a) is loose clay having a density of 100 pcf
Soil (b) is lean, stiff clay having a density of 127.8 pcf, an unconfined compression strength of 2.0 tsf and a cohesion of 2000 psf
Depth D_w to ground water is 10′ –0″
Allowable settlement is 1½″. Settlement analysis will include only those soils within the 10% pressure bulb. The height H in Formula (6-2), therefore, will be taken as twice the width of the footing—16 feet.

Example 6-3

Required: To determine the allowable soil bearing pressure and the footing capacity in accordance with the procedures of Chapter 5.

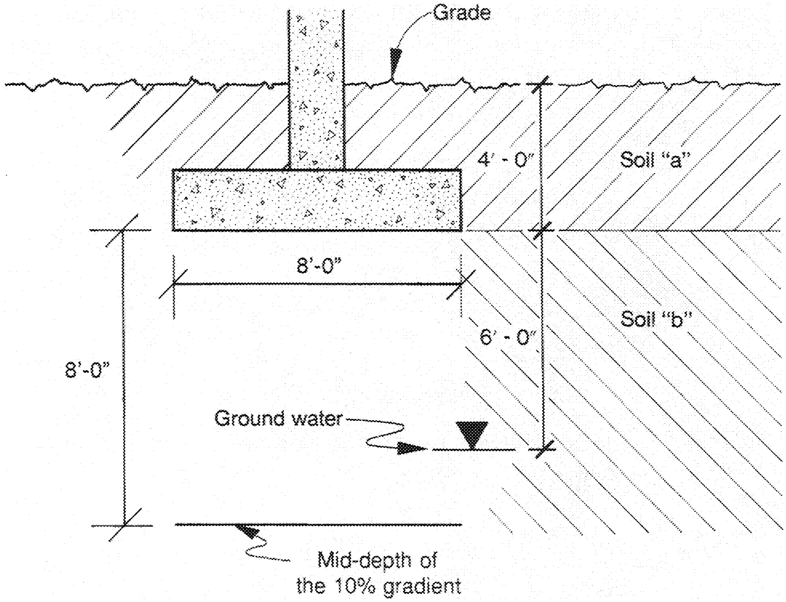


FIGURE 6-9. Examples 6-3 to 6-6—General conditions.

Bearing capacity factors for clay: $N_c = 5.14$ $N_q = 1.0$ $N_\gamma = 0$

From Figure 5-5: The water table is located in level 4.

When applying Equation (5-2), water at this level will not effect the second term. Water would normally effect the third term, but in a clay soil the third term is deleted.

From Equation (5-2): $q_d = 1.2 \times 2000 \times 5.14 + 100 \times 4.0 \times 1.0 = 12{,}736$ psf

From Formula (5-5): $q_a = \dfrac{12736 - 100 \times 4.0}{3} = 4{,}112$ psf

The maximum safe load for which this footing may be designed is:

$$P = 4.112 \times 8.0 \times 8.0 = 263 \text{ kips}$$

Example 6-4

Required: To compute values for V_s, V_v, and e_o, and then to draw the e-log p curve for the lean, stiff clay, based on the results of a consolidation test.

Additional information supplied by the testing laboratory:

$$\gamma_{dry} = 106.4 \text{ pcf} \qquad G_s = 2.68$$

By the procedures outlined in Article 6-4:

$$\text{Volume of solids } V_s \text{ in 1 cubic foot} = \frac{106.4}{62.4 \times 2.68} = 0.6362 \text{ CF}$$

$$\text{Volume of voids } V_v \text{ in 1 cubic foot} = 1.0000 - 0.6362 = 0.3638 \text{ CF}$$

$$e = \frac{0.3638}{0.6362} = 0.572 = e_o$$

Note that the void ratio of the in-place soil within the 10% gradient line is also the void ratio e_o in the sample at the start of the consolidation test.

A consolidation test was performed on a sample of this material. The reduction in height of the sample under different loading conditions is used to determine the variation in void ratio and hence, the e-log p curve. The results of this test are given in Table 6-4.

The initial height of the sample used in the test was 1.0000 inches. Therefore, the equivalent height of solids is:

$$h_s = 1.0000 \times 0.6362 = 0.6362 \text{ inches}$$

The following is a sample of the way in which void ratio is computed:

$$e \text{ (at 8 tsf)} = \frac{\text{height of voids}}{\text{height of solids}} = \frac{0.9256 - 0.6362}{0.6362} = 0.455$$

This calculation determines one point on the e-log p curve. Similar calculations define the entire curve as shown in Figure 6-10.

TABLE 6-4. Example 6-4—Consolidation Test Data

Load (tsf)	Dial Reading at Start	Dial Reading at Finish	Consolidation 0.0001″	Sample Height	Void Ratio
0				1.0000	0.572
½	3	21	18	0.9982	0.569
1	8	66	58	0.9924	0.560
2	10	143	133	0.9791	0.539
4	19	267	248	0.9543	0.500
8	7	294	287	0.9256	0.455
16	2	301	299	0.8957	0.408

Note: These void ratios, when plotted against their corresponding loads, produce the e-log p curve shown in Figure 6-11.

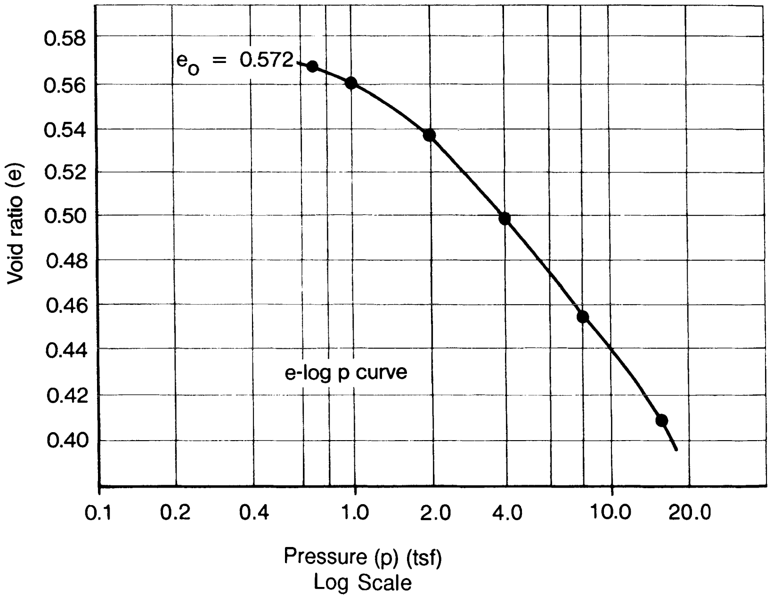


FIGURE 6-10. Example 6-4—The *e*-log *p* curve.

Example 6-5

Required:

1. Compute the overburden pressure p_o existing at the mid-depth of the specified pressure bulb, as defined in Figure 6-9. Because this is a square footing, the pressure bulb extends 16 feet below the footing, in accordance with Figure 5-4.
2. Compute the pressure Δp induced by the superimposed load on the footing.
3. Compute the resultant pressure p_f.

Calculations:

1. The overburden pressure computed at the mid-depth of the 10% gradient line is:

$$p_o = 100 \times 4.0 + 127.8 \times 6.0 + (127.8 - 62.4)\ 2.0 = 1298 \text{ psf}$$

2. The pressure Δp induced by the load of the footing will be computed by using each of the three procedures previously introduced in Article 6-5. The reader can then make a comparison:
   (a) Draw a horizontal line in Figure 5-3 midway between the base of the footing and the 10% gradient line. Then draw two vertical lines down to this horizonal line. The first from the edge of the footing and the second from the center of the footing. Next, read the pressure gradient

where these two lines intersect the horizontal. The increase in pressure can be taken as the average of these two gradients times the pressure intensity directly beneath the footing. By scale, then:

$$\Delta p = \frac{0.20 + 0.32}{2} \times 4112 = 1069 \text{ psf}$$

(b) The increase in pressure computed in accordance with Figure 6-8 is:

$$\Delta p = \frac{263{,}000}{(8 + 8)\,(8 + 8)} = 1027 \text{ psf}$$

(c) Using Appendix C, the increase in pressure is taken as the average of the pressures induced at the corner and center of the footing.

At the corner, according to Article C-4: $m = n = \frac{8}{8} = 1.0$

In the above, the numerator is the side of the footing, the denominator is the distance to the mid-depth of the 10% gradient.

Therefore: $C_3 = 0.175$

and $p_v = 0.175 \times 4112 = 720$ psf

At the center, according to Article C-5: $m = n = \frac{4}{8} = 0.5$

Therefore: $C_3 = 0.085$ for each of four squares.

and $p_v = 4 \times 0.085 \times 4112 = 1398$ psf

The average induced pressure Δp is: $\frac{720 + 1398}{2} = 1059$ psf

3. The three methods used in the preceding calculations give essentially the same result. Experience indicates that this will usually be the case. Method (a) differs from the other two in that the calculations related to a slightly higher plane. This is because the actual depth of the pressure bulb is somewhat less than the assumed depth of two times the width of footing. There would be a dramatic difference, however, in the case of a continuous footing, in which case method (a) should be used. In this instance, method (b) will be adopted. Therefore:

$$p_f = 1298 + 1027 = 2325 \text{ psf} = 1.163 \text{ tsf}$$

Example 6-6

Required: To obtain the value of e_f from the e-log p curve and then to compute the anticipated settlement of the subject footing shown in

Figure 6-9. Finally, to reduce the footing capacity if settlement exceeds the allowable.

It is first necessary to determine the void ratio e_f corresponding to p_f. This may be read off of the e-log p curve, or it may be computed. The value of p_f, plotted as 1.623 tsf, falls between the test loads of 1.0 tsf and 2.0 tsf. An enlargement of the curve within that range is shown in Figure 6-11. Assuming the e-log p curve to approximate a straight line, the value of e_f may be found by similar triangles.

$$\frac{e_f - 0.539}{0.560 - 0.539} = \frac{\log 2 - \log 1.163}{\log 2 - \log 1}$$

From which: $e_f = 0.555$

It should be noted that this value of e_f is on the conservative side since the actual curve rises somewhat above a straight line. This will result in computed settlements slightly higher than those actually realized. The anticipated settlement is:

From Formula (6-2): $\Delta = \left[\dfrac{0.572 - 0.555}{1 + 0.572}\right] 16 \times 12 = 2.08$ inches

This settlement exceeds the specified allowable of one and one-half inches. The values of q_a and P must be reduced as follows:

$$q_a = \left[\frac{1.50}{2.08}\right] 4{,}112 = 2{,}956 \text{ psf} \qquad P = \left[\frac{1.50}{2.08}\right] 263 = 189 \text{ kips}$$

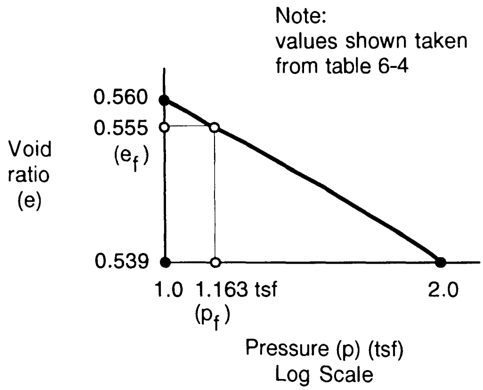


FIGURE 6-11. Example 6-6—The e-log p curve from Figure 6-10, enlarged.

7

Spread Footings

7-1. GENERAL

Spread footings are generally a large pad of reinforced concrete poured into an excavation which has been dug by hand or by machine, but which has been trimmed by hand to specified dimensions. The primary purpose of a footing is to transfer load from the building to the ground. The load may be vertical, as in the case of gravity, or it may be horizontal, as in the case of wind, earthquake or earth pressure. A secondary purpose of a footing is to provide a base upon which formwork may be erected.

Footings supporting walls are usually continuous. Footings supporting columns are usually square or rectangular and may be designed to support individual or multiple columns. Footing selection depends upon the architectural layout of the columns with respect to each other and with respect to property lines or other obstructions.

7-2. FOOTING EXCAVATION

Earth Formed Footings

Whenever possible, the footing should be poured into an excavation whose side walls are primarily the original, undisturbed earth. This requires care on the part of the contractor particularly since the main basement excavation will be made by machine. After the area around the footing has been cleared, wood forms called screeds or screed rails are usually installed at the top of the footing. These forms, which extend around the four sides of the footing, serve the following purposes:

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

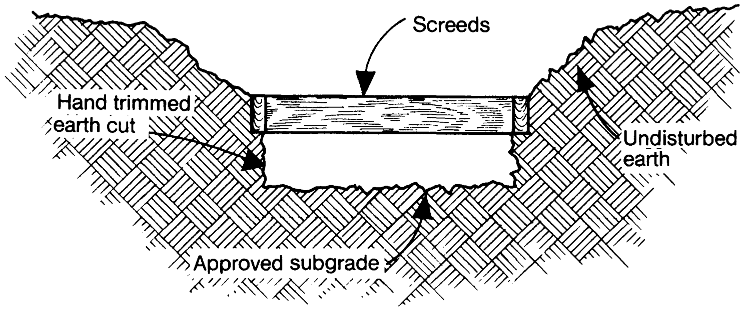


FIGURE 7-1. Preparation for footings poured in earth forms.

1. They physically identify the elevation, extent and width of the footing and provide a continuous shelf to which the footing concrete shall be cast.
2. Experience shows that the upper edges of almost all earth forms are very susceptible to cave-in. The screeds stabilize these edges and effectively prevent most cave-ins.

A typical earth formed footing detail is shown in Figure 7-1. This kind of installation is usually very practical for use in cohesive soils. The side walls of an excavation cut into a cohesive soil will remain stable for some period of time without being supported by formwork. The concreting of the footing must proceed expeditiously, however, because the side walls will cave in if abused by workmen or if subjected to heavy rainfall.

Wood Formed Footings

Granular soils differ from cohesive soils in that the side walls of an excavation will not stand vertically, even for a short while. The presence of a limited amount of fines may lead to the erroneous conclusion that the soil possesses sufficient cohesion for the side walls to stand without support. Such soils must be carefully tested at the site to insure that adequate forming procedures will be used. In soils whose side walls will fail, the footing area must be over-excavated and a wood formed box must be constructed for the full height of the footing. A typical wood formed footing detail is shown in Figure 7-2.

After the footing has been poured and the forms removed, the area of over-excavation must be filled. This can be accomplished in either of the following ways:

1. Fill the area with lean concrete.
2. Backfill the area with approved soil compacted to the specified density.

7-3. APPROVAL OF SUBGRADE

All footings must extend down to, and bear on a subgrade inspected and approved by the architect or engineer of the project. The allowable bearing pressure of

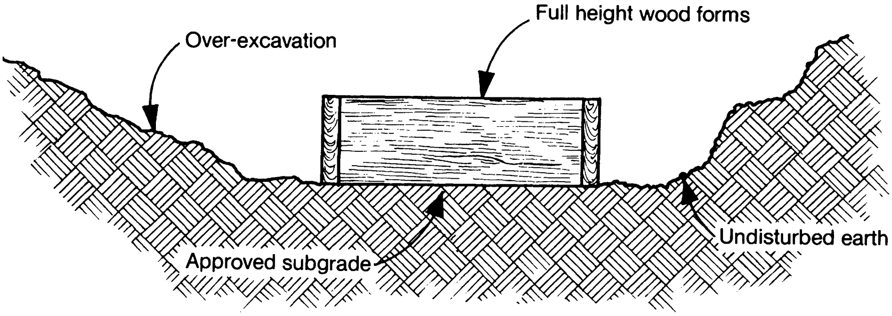


FIGURE 7-2. Preparation for footings poured in wood forms.

this subgrade must be consistent with that for which the footings were designed during the design stage of the project.

The elevation of the bottom of the footings must be specified on the contract drawings. During the excavation process it is possible that soil having an acceptable bearing pressure may be found above the specified elevation. In this instance the footing may be raised, subject to engineering approval and provided that raising the footing will not cause interference with any other construction. When acceptable bearing pressure is not found at the specified elevation, the footing must be lowered accordingly. Unit prices by which the owner and contractor can both be equitably compensated for a change in footing elevation are commonly a part of the general contract.

The bottom of the footing must be reasonably leveled and cleared of all debris, trash, loose stones, etc. Holes left after the removal of loose stones shall be filled with footing concrete when the footing is poured.

7-4. LOAD TRANSFER BETWEEN FOOTING AND GROUND

Footings distribute their load into the ground through direct bearing on the surface of contact. The required area of contact is computed from the following formula:

$$q_a = \frac{P}{B \times D} \tag{7-1}$$

In which:

q_a = the allowable soil bearing pressure as determined by the principles set forth in Chapters 5 and 6

P = the load supported by the footing plus the weight of the footing itself

B and D = the width and length of the footing

Note: When the footing is continuous, as under a wall, the length of footing is taken as one linear foot

7-5. TYPICAL FOOTING DETAILS

Minimum Footing Width

The footing widths given in Figure 7-3 are considered a reasonable minimum, based primarily on facilitating construction.

Wall Footings

Wall footings are continuous footings that not only transfer vertical and horizontal loads from the wall into the soil, but also provide a shelf upon which to erect forms for the construction of the wall. Where the wall is discontinuous for short lengths, as in the case of doors, windows or other openings, the footing is usually poured as a continuous member without regard to those openings.

Wall footings must sometimes be stepped down from their normal elevation due to the presence of poor soil or the close proximity of adjacent, lower footings. This is illustrated in Figure 7-4, in which the dashed line indicates the angle of proximity. In the interest of time and costs, the contractor will generally want to use a relatively steep angle, but this should only be allowed based on an engineering evaluation. The maximum value of this angle is limited by the angle

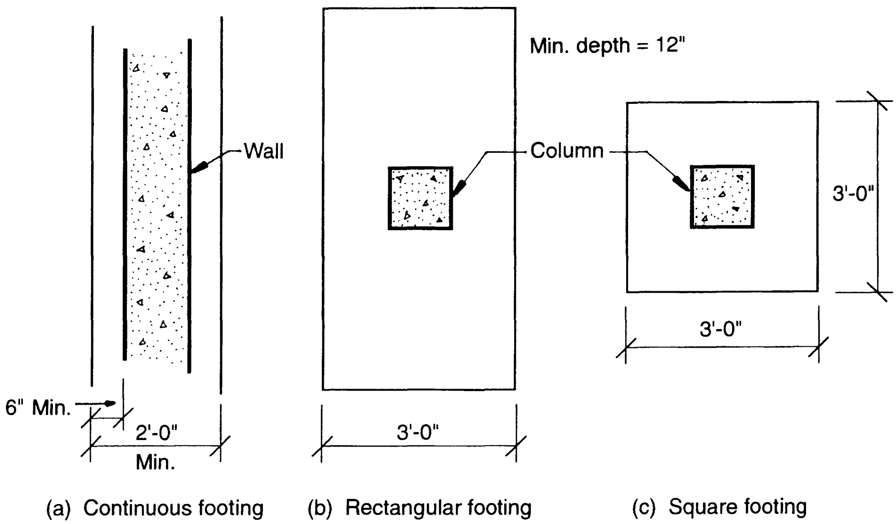


FIGURE 7-3. Recommended minimum widths of different footings for ease of construction.

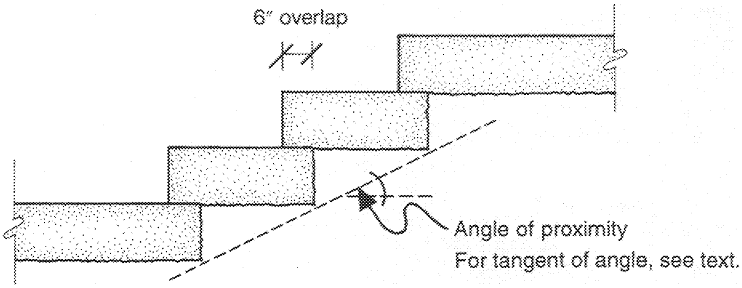


FIGURE 7-4. Recommended angle of proximity for stepped footings.

of repose as described in Article 10-1. Without engineering evaluation the tangent of the angle of proximity should not exceed the following:

1. In cohesive soil—1 : 2
2. In granular soil—1 : 3

Particular care must be taken by the contractor to maintain vertical or near vertical cuts when excavating for stepped footings.

Individual Column Footings

It is important for the center of gravity of the supported load to coincide with the centroid of the footing, and this should be done wherever possible. The soil bearing pressure is then uniformly distributed as illustrated in Figure 7-5(a). When this condition is not met, the soil bearing pressure will no longer be uniformly distributed because of the resultant eccentricity as illustrated in Figure 7-5(b). An eccentrically loaded footing can lead to serious consequences.

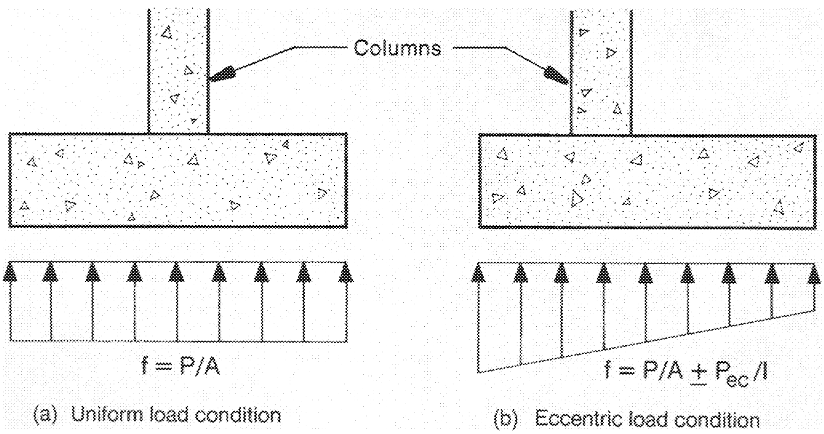


FIGURE 7-5. Difference in soil pressure distribution between footings supporting concentric and eccentric loads.

Although a certain amount of eccentricity can be tolerated, as in the case of an exterior retaining wall where bearing stresses are kept very low, it should be avoided wherever possible. The middle third rule should remind us that if the center of gravity of the load were to fall outside of the middle third of the footing then the bearing pressure on the opposite edge of the footing would be negative. Since tension cannot be developed on the surface of contact between the footing and the soil, failure could occur.

If a footing is overexcavated and then filled with footing concrete, it might be inferred that the footing was eccentrically loaded. There is no harm, however, in pouring the overexcavation along with the footing as long as the required size of footing is centered beneath the column.

Combined Footings

There are times when two columns are so close together that they must bear on a common footing. This condition is illustrated in Figure 7-6. These columns cannot be supported on individual footings because the footings would overlap. Rather than use long, slender rectangular footings a combined footing will be used. The particular feature of this type of footing is the need to position the centroid of the footing directly beneath the center of gravity of the loads. The reason for this, of course, is to avoid eccentricity.

The clear distance between the edge of the footing and the nearest face of the column should be given careful consideration by the engineer. In the absence of any special conditions the author recommends a minimum clear distance equal to the width of the column or 12 inches, whichever is the larger.

Mat Foundations

There are occasions when it becomes necessary or advantageous to support a large number of columns on a common footing. Such a footing is commonly

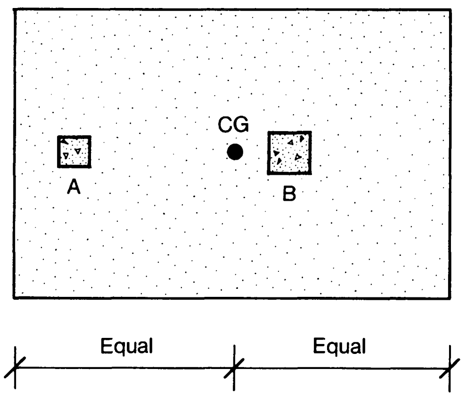


FIGURE 7-6. Typical combined footing. Note location of center of gravity.

called a mat or raft foundation. A mat foundation is usually considered to be a good engineering response to either of the two following situations:

1. The close proximity of a large number of columns, which would result in a considerable amount of overlapping of individual footings.
2. The existence of poor soil at normal bearing elevations, which would not only require the use of large, overlapping footings, but which would also raise the question of serious differential settlement.

Mat foundations are usually constructed as an integral part of the basement floor. They are usually at least 24 inches in thickness and are heavily reinforced. Design is essentially that of an inverted flat plate.

7-6. TYPICAL FOOTING REINFORCEMENT

Column Footings

Footings supporting columns distribute their load into the ground by two-way action. This is to say that the footing is subjected to bending stresses along each of its two axes. Tensile reinforcing, therefore, is required in each direction. For footings carrying a single column reinforcement is required only at the bottom of the footing. For footings carrying multiple columns, reinforcement is also required at the top of the footing. These requirements are illustrated in Figure 7-7, where the bending moment induced by the soil bearing pressure is plotted on the compression side of the footing.

Reinforcing shall be placed in both directions on the tension sides of the footing, as indicated in the same figure. Bars shall be spaced equally throughout

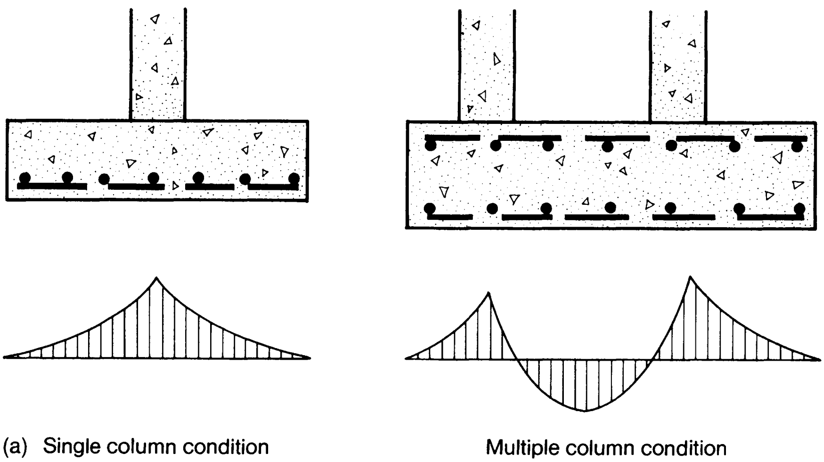


FIGURE 7-7. Typical reinforcing arrangement in footings for single and multiple column conditions.

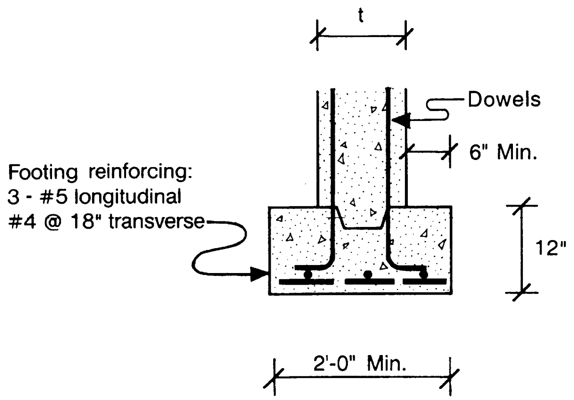


FIGURE 7-8. Recommended minimum size and reinforcement for wall footings.

the width of the footing, with the first bar approximately 3 inches clear from the footing edge. Bars shall be wired together at sufficient intersections so as to produce a rigid mat of bars. The lower mat shall be supported to the proper elevation by pieces of brick, block or precast units made for that purpose. Metal supports, pushed into the ground, should not be permitted. The clear concrete cover over all reinforcement should be no less than three inches for the lower reinforcing and two inches for the upper reinforcing. This amount of cover is recommended as a means of protecting the reinforcing from corrosion by the action of soil, water, or air.

Wall Footings

Wall footings transfer their load to the soil linearly. Bending, therefore, occurs around only one axis. Under normal conditions, the bending stresses in these footings is quite low. For ease of construction and to provide a degree of safety against the unexpected, all footings, even those which are lightly loaded, should meet certain minimum requirements as to size and reinforcing. Recommendations regarding these requirements are given in Figure 7-8.

Note that the shear key in this detail is sloped on both sides. This is the way preferred by contractors because it facilitates removal of the tapered wood form key. In those instances when the wall must transfer lateral load to the footing, the side transferring the load must be vertical. Shear keys are discussed in detail in Article 11-7 and Appendix B.

7-7. VERTICAL DOWELS

Column Dowels

All columns are reinforced with vertical bars, whose purpose is to carry a proportionate share of the load. The force carried by these bars must be transferred into the footing concrete. This can be accomplished in either of following three ways:

1. *Positioning the vertical bars in the footing before pouring footing concrete:* Because of the difficulty of holding long vertical bars in place without the benefit of formwork, this method is limited to relatively short columns and subsequently short reinforcing bars.
2. *Using a short piece of vertical bar called a dowel:* One dowel is required for each column bar and is positioned in the footing prior to the pouring of footing concrete. After the footing has been poured each column bar is then placed in hard contact with a dowel and wired to it. The force in the column bar is then theoretically transferred into the dowel and from the dowel into the footing. The use of dowels to transfer load from a column to a footing is illustrated in Figure 7-9. Note that the dowels must be carefully located in the footing so that the column bars, which are placed adjacent to them, will be accurately positioned in the column. Also note that the use of dowels can cause considerable crowding of reinforcement, thereby complicating the job of proper concreting. For this reason, this method is usually restricted to columns having relatively small bars, #7 or less, and to situations where there are not enough bars to cause crowding. In other situations bars should be welded, as described in item 3.
3. *Sitting the column bars directly onto the dowels and welding them:* This gives positive continuity to the reinforcing system and avoids bar crowding. Each dowel must be accurately positioned to receive the column bar. The bearing surface of the lower bar should be cut straight across. The surface of the upper bar should be beveled in the fabricating shop. This is necessary to insure proper access to the welding electrodes. The upper bar should be oriented so that the bevel is positioned outward and, therefore, more accessible to the welder. All bars must be rigidly held in position during the welding process. This method can be used for any size and arrangement

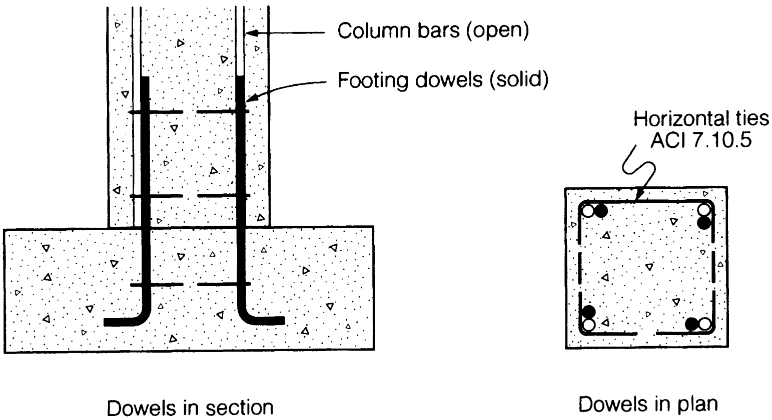


FIGURE 7-9. Dowel placement in column footings.

of bars. Its use is particularly encouraged in heavily reinforced columns or in columns having large bars.

Wall Dowels

Wall dowels are similar to column dowels except that they are usually smaller in diameter and have more space between adjacent bars. This arrangement lends itself very well to the use of lapped dowels.

Code Requirements Relating to Dowels

All requirements relative to dowels, including development lengths, laps, and bar substitutions, are given in Appendix E.

7-8. PROPERTY LINE CONSIDERATIONS

There are times when the architectural layout may position columns close to the property line. Building codes usually do not permit extension of any part of the building beyond that line. Some building codes even require a set back from the building line. In such instances, there may not be sufficient space in which to position a square footing. Three alternative footing arrangements must then be considered.

1. A rectangular footing
2. A strap footing
3. A combined footing

The use of these alternatives is discussed in the following paragraphs.

Rectangular Footings

A typical arrangement of a rectangular footing adjacent to a property line is indicated in Figure 7-10.

The width and length of the footing shall be computed as follows:

$$B = 2(x - y) \qquad \text{and} \qquad D = \frac{P}{Bq_a}$$

Rectangular footings should be proportioned so that the length to width ratio does not exceed three. It should be noted that a rectangular footing becomes more of a one-way element as the length to width ratio increases. This type of footing will therefore be thicker, more heavily reinforced and less cost effective.

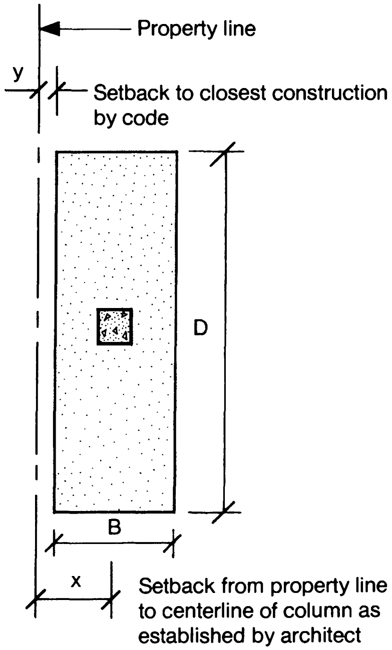


FIGURE 7-10. The use of a rectangular footing for columns adjacent to a property line.

Strap Footings

There are instances when rectangular footings cannot solve the problem of the close proximity of a column to a property line. Subject to the architectural layout of the adjacent columns, this problem can be solved by the use of a strap footing, as illustrated in Figure 7-11.

If it were not for the strap, the offset footing would be subjected to an undesirable eccentric load, which would induce a potentially hazardous distribution of stress into the soil beneath. This condition was previously indicated in Figure 7-5. The strap acts as a large overhanging beam and transfers the load of the exterior column back to the centerline of the offset footing. The offset footing, therefore, is subjected only to an axial load and the soil beneath to a uniformly distributed load. It is to be expected that the strap will be heavily reinforced with top bars due to the large bending moment induced by the property line column. Note that the offset footing will be somewhat larger than the rectangular footing of Figure 7-10 because of the increase in axial load due to the cantilever action of the strap.

The vertical positioning of the strap is optional, as indicated below:

1. The strap and the footing can be flush top, in which case the strap and footing must be poured monolithically.
2. The strap can sit directly on the footings, in which case it can be poured at any time following the footing pour.

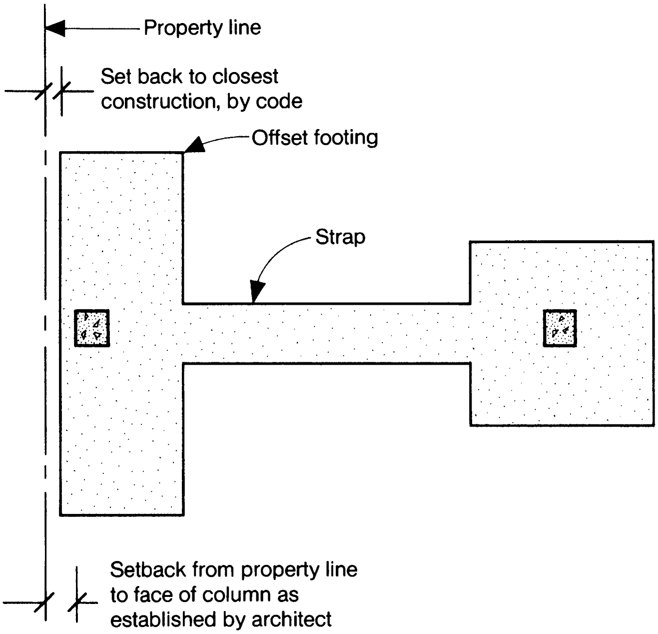


FIGURE 7-11. The use of a strap footing for columns adjacent to a property line.

The decision as to which way to go should logically be that of the contractor, who is actually going to perform the work. The engineer, however, must design and detail the strap prior to the work going out for bids. After the contract has been awarded the contractor could be offered the option of submitting a different design for approval, provided that the contract price would not increase.

Combined Footings

A combined footing, similar to the one previously illustrated in Figure 7-6 can be used to solve the property line dilemma, provided that it works dimensionally. This requires that the centroid of a reasonably proportioned footing can be positioned directly beneath the center of gravity of the loads.

7-9. FACTORS AFFECTING VERTICAL PLACEMENT OF FOOTINGS

Acceptable Soil Bearing Pressure

Footings, of course, must bear on acceptable soil. What is acceptable for one building, however, may not be acceptable for another. A multi-story concrete building weighs considerably more than a two or three story lightweight steel frame building. It is the nature of soil that its load bearing capacity increases with depth. The footings of the multi-story building, therefore, can be expected to be placed at a lower elevation than those of the light weight steel frame. Heavily loaded footings could be placed at lesser depth, with a corresponding increase in size to account for the decrease in soil bearing capacity. However, this is not considered to be good engineering practice. The general rule is that large loads should bear on large capacity soil. Exceptions to this rule must be based upon proven footing performance and a substantial reduction in cost. Settlement must also be considered. Weaker soils compact more than dense soils, and even though the footing on the weaker soil must be larger, it will still settle more.

For a discussion of soil bearing pressure, refer to Chapter 5.

Placement With Respect to Frost

All footings, both interior and exterior, must be protected against the adverse action of frost. All earth contains water, and when water freezes it expands with an almost irresistible force. When the earth directly beneath a footing freezes, the footing will be pushed upward. Considerable damage, and possibly catastrophic failure, may be the end result.

Footings, therefore, must be placed below the frost line. The depth of this line below the surface of the ground is a function of locality. In order to complete his design, the designer must obtain definitive information regarding the frost line depth from the building officials of the locality in which the structure is to be built. The information given in Figure 7-12 may be used to gain insight as

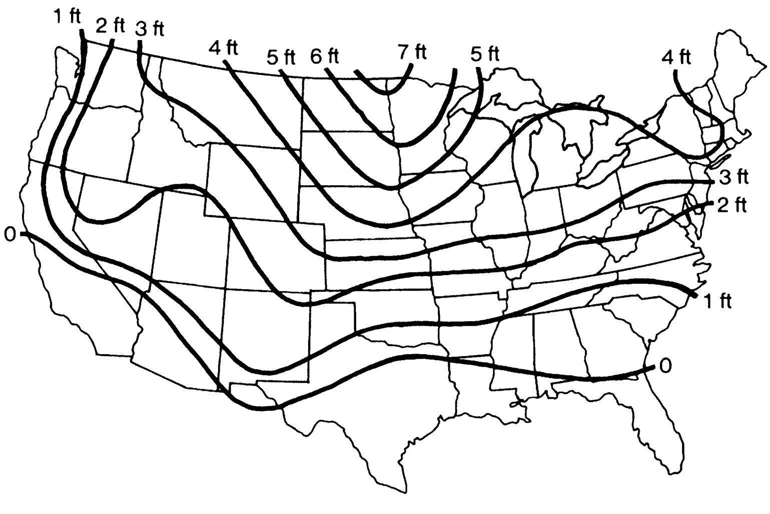


FIGURE 7-12. Maximum anticipated depths of freezing as obtained from city building codes. Actual depths may vary considerably, depending on ground cover, soil, moisture, topography, and weather. [Ref. 18]

to the general variation of the frost line depth in different areas of the country. This information, however, should not be used for design without proper verification.

Placement With Respect to Expansive Soil

The presence of expansive soil and the variation in plasticity index with depth can only be determined accurately by laboratory analysis. It is the opinion of the author, as stated in Article 13-9, that building foundations should not be constructed on soil having a plasticity index greater than 10. With the acceptance of this as a premise, construction of the foundations should then proceed in one of the following ways:

1. Excavate through the expansive soil and install spread footings on good bearing soil beneath. When the amount of excavation required to obtain good bearing is no more than 10 to 15 feet, contractors usually prefer this method as being faster and more cost effective.
2. Drive piles or drill piers through the expansive soil and extend them to the more resistant material normally found at greater depth.

Note: For information regarding the construction of slabs on expansive soil, refer to Article 13-9.

Proximity To Adjacent Footings

The elevation of adjacent footings must be established so that lower level footings do not undercut footings placed at an upper level. This problem, as illustrated in Figure 7-13, is essentially the same problem previously discussed in regards to the stepping of wall footings, as illustrated in Figure 7-4. Without benefit of an engineering evaluation, the tangent of the angle of proximity is usually limited to 1 : 2 in cohesive soils and 1 : 3 in granular soils. When excavating in rock or in very dense clay, it may be possible, subject to an engineering evaluation, to increase the slope to 1 : 1.

It is noted that the angle of proximity is drawn from the top of the lower

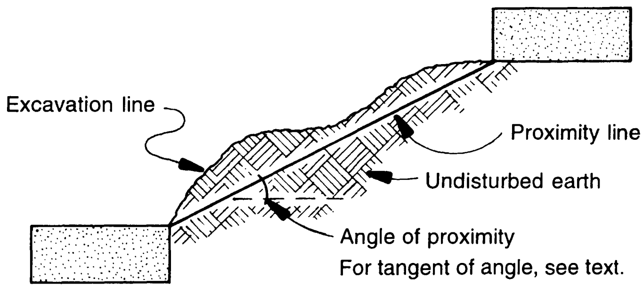


FIGURE 7-13. Recommended angle of proximity between adjacent footings of different elevations.

footing to the bottom of the upper footing. This condition is only valid when the lower footing is poured in earth forms. When the lower footing is poured in wood forms, the angle of proximity should be drawn from the bottom of the lower footing to the bottom of the upper footing.

These recommendations are only valid providing that the line of excavation does not undercut the proximity line. If undercutting occurs, a new proximity line must be drawn from the lower footing up to the underside of the upper footing and flattened as necessary to insure that all undercutting is above that line.

Proximity to Adjacent Properties

It should be recognized that the architectural layout of a building may be severely limited or curtailed due to the presence of an existing building on the adjacent property.

When there is an existing building on the adjacent property, a survey should be made to determine the location and depth of all footings near the property line. Adequate separation must be provided between the existing and new footings. This is particularly important in those instances when the new footings are to be placed lower than the existing ones. The angle of proximity as shown in Figure 7-13 may serve as a starting point, but the final slope must be the result of an engineering evaluation.

When excavating close to adjacent property, care must be taken to assure that the adjacent property will not sustain any damage whatsoever, either during or after construction. Damage could be attributable to any of the following causes: cave-in, settlement, disturbance during construction such as vibration, blasting, etc., and lowering of the existing water table. Any kind of damage may become the basis for legal action, which may delay or suspend construction. Such action would certainly be costly to defend and to correct, if those in charge of the new project are found at fault.

Effect of Ground Water

The existence of ground water can be determined during the conduct of the subsurface soil exploration, as described in Article 3-6. This exploration, of course, should be made well in advance of a design commitment by the architect. With a high water table the architect may find that his preliminary design of the building must undergo extensive change, particularly in the case of basements or underground facilities.

Major construction below the water table is both difficult and expensive. It is one thing to have footings below the water table. It is an entirely different matter to have a part of a building below the water table. This latter condition can have serious architectural consequences. It is for these reasons that construction at or below the water table must be very carefully thought out during the design stage of the project.

Structures built below the water table are subjected to hydrostatic pressure. In lightly loaded buildings this uplifting pressure can become a formidable obstacle. Note also the serious consequences in time and costs due to the need for waterproofing all areas of the building which are below the highest anticipated water table.

In some instances it may be possible to temporarily drain the area, thus lowering the water table so that construction can proceed in the dry. Before adopting this procedure, however, a study must be made to ascertain the short term and long term effects that this procedure will have on this property and on any adjacent properties.

When it is required to do so, footings can be poured under water using a tremie or elephant trunk, as described in Article 8-20.

The existence of ground water implies a softer soil. It may be possible to bypass the water problem entirely by installing piles or piers instead of spread footings.

7-10. DEAD LOAD BEARING PRESSURE

Engineers have long recognized the principle of equal dead load bearing pressure. The meaning of this is that the bearing area of all footings should be adjusted so that the bearing pressure due solely to the action of dead load would be essentially equal for all footings. The size of each footing, of course, must be adequate to satisfy the allowable soil bearing pressure established for dead load plus live load.

When footing sizes are adjusted to provide equal dead load pressure, it is evident that there may be considerable variation between footings in pressure due to total load. This is normally not considered to be a problem because in most buildings there is very little actual live load.

For application of this principle, refer to Example 7-5.

7-11. CLOSING RECOMMENDATIONS

As noted in Chapter 5, it is very difficult to predict with reasonable accuracy the way in which a footing will respond to the loads imposed upon it. Experience indicates the need for extra caution in the design of the foundations of a building. A troubled footing cannot be easily repaired.

Construction below grade is considerably more difficult than construction above grade and, when deep in the excavation, there is an inherent need for haste. Weather can play havoc with construction, particularly when below grade. Rain, snow, and even wind can delay the work for days on end and can cause additional expenditure of time and money. Nowhere are these problems more evident than in the construction of foundations. The designer should recognize these problems and, wherever possible, should direct his design and construction schedule to give the contractor every opportunity to use his capabilities to the best advantage.

There is no substitute for experience. It is strongly recommended, therefore, that the architect and engineer consult with a general contractor who does work in the general locality of the building site. He will be able to advise as to what construction techniques are suitable for the site under consideration. The advice and counsel of a knowledgeable contractor, when consulted at an early stage of the design process, can prove invaluable to the success of the project.

7-12. SAMPLE PROBLEMS

In all problems it is assumed that the superimposed load carried by the footing has been increased 5% to account for the weight of the footing. Under normal circumstances this is a reasonable percentage of increase. This method is one of the several methods by which the weight of the footing may be accounted for in design.

Regarding footing sizes, it is the custom to specify footing widths in increments of 3 to 6 inches for relatively small footings, and 6 to 12 inches for larger ones.

Example 7-1

Required: (1) To determine the required area of contact between a spread footing carrying a superimposed load of 250 kips, and earth having an allowable soil bearing pressure of 3 tsf.
(2) To recommend the size footing.

Since there are no apparent restrictions to the proportioning of this footing, it will be made square. The footing design will therefore use the two-way action, which is inherent in most concrete construction. A square footing is also preferable from the standpoint of cost.

From Formula (7-1):

$$6.0 = \frac{250}{A}$$

$$(\textit{Note } q_a = 6.0 \text{ ksf})$$

Therefore: $A = 41.7$ square feet and $B = 6.45$ feet

And finally: $B = 6'\text{-}6''$

Example 7-2

Required: To determine the length and width of a combined footing to carry the columns indicated in Figure 7-14. The allowable soil bearing pressure is 3 tsf.

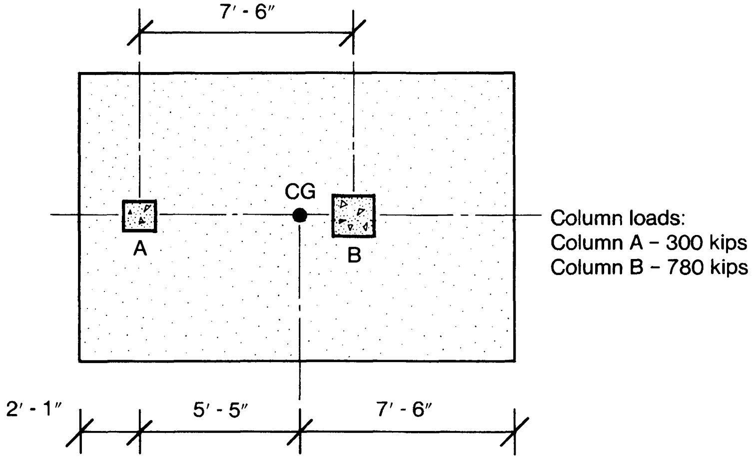


FIGURE 7-14. Example 7-2—Combined footing.

If these columns were supported on individual footings, it can be seen that the footings, if square, would overlap. Rather than use long and narrow rectangular footings, a combined footing will be used.

In order to avoid unwanted eccentricity, the centroid of the footing must coincide with the center of gravity of the loads. This center, measured from column A, is calculated by taking moments about column A:

$$x = \frac{780 \times 7.5}{300 + 780} = 5.42 \text{ feet}$$

From Formula (7-1):

$$6.0 = \frac{1080}{A}$$

Therefore: $A = 180$ square feet and a 12′-0″ × 15′-0″ footing may be used. Note that any number of footing sizes would satisfy area and clearance requirements.

Example 7-3

Required: To determine the size of a rectangular footing required to carry a column adjacent to a property line, as shown in Figure 7-15.

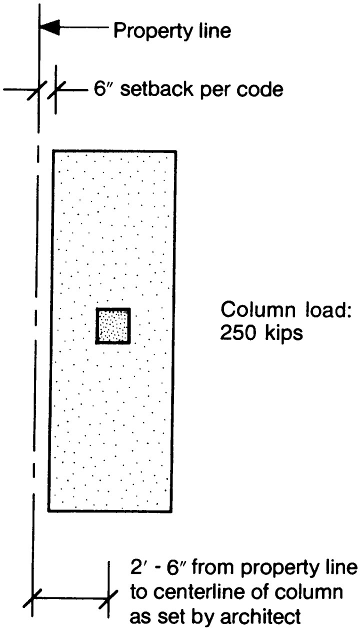


FIGURE 7-15. Example 7-3—Rectangular footing.

The column load is 250 kips, and the allowable soil bearing pressure is 3 tsf.

The footing width is computed as: $B = 2(2.5 - 0.5) = 4.0$ feet

From Formula (7-1):

$$6.0 = \frac{250}{4.0 \times D}$$

From which: $D = 10.4$ feet, and the final footing size is 4′-0″ × 10′-6″.

Example 7-4

Required: To redesign Example 7-3 using a strap footing assembly, as detailed in Figure 7-16. The load on column B is 400 kips.

First: Assume a 4′-0″ wide footing, then calculate the resultant footing load. Note that the strap acts as an overhanging beam.

$$\left[\frac{16.0}{15.0}\right] 250 = 267 \text{ kips}$$

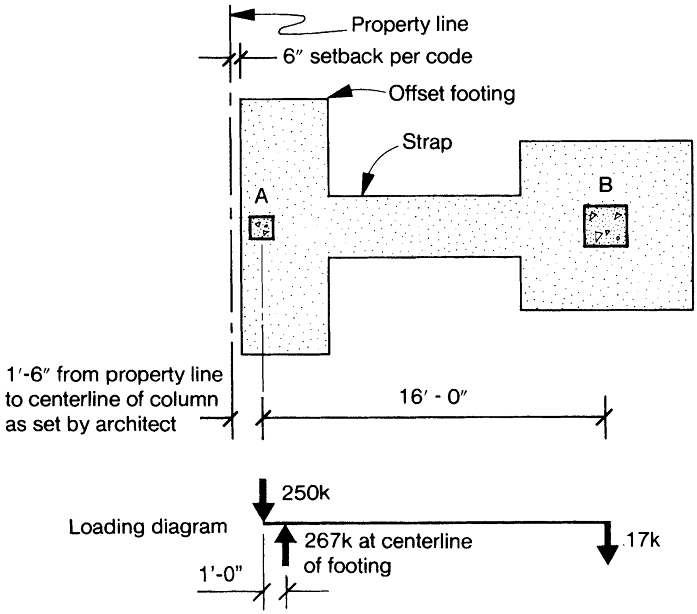


FIGURE 7-16. Example 7-4—Strap footing.

From Formula (7-1):

$$6.0 = \frac{267}{B \times D} \qquad \text{Use } 4'\text{-}0'' \times 11'\text{-}6'' \qquad \text{at column A}$$

$$6.0 = \frac{400}{B \times B} \qquad \text{Use } 8'\text{-}6'' \times 8'\text{-}6'' \qquad \text{at column B}$$

As anticipated, the strap footing is somewhat larger than the rectangular footing it replaced.

Example 7-5

Required: To adjust the sizes of the footings listed in Table 7-1, so that each footing will have essentially the same dead load bearing pressure. The presumptive bearing pressure for which the footings must be designed is 4 tons per square foot. Footings shall be sized in three inch increments.

The solution to this problem requires four steps.

1. To determine the minimum footing area that will satisfy the total load for each column.

TABLE 7-1. Example 7-5—Design Loads for Footings

Footing	Dead	Live	Total[a]
A	240	150	390
B	350	140	490
C	600	440	1040
D	410	160	570

[a] All loads are in kips.

2. To calculate the dead load bearing pressure for each footing by dividing the given dead load by the area found in step 1.
3. To calculate the footing area required to equalize the dead load bearing pressure under all of the footings. This is done by dividing the dead load of each footing by the least dead load bearing pressure found in step 2.
4. To determine the footing size based on the area computed in step 3. Sizes are selected in 3-inch increments.

The results of this procedure is tabulated in Table 7-2. Sample calculations for Footing #1 are included herein:

For Footing A:

(1) $\dfrac{390}{8} = 48.75$ square feet (*Note:* 4 tsf = 8 ksf)

(2) $\dfrac{240}{48.75} = 4.92$ ksf

(3) $\dfrac{240}{4.62} = 51.9$ square feet

(4) $\sqrt{51.9} = 7.21$ feet—use 7′-3″

TABLE 7-2. Example 7-5—Footing Design Based on Dead Load Bearing Pressure

Footing	(1)	(2)	(3)	(4)
A	48.75	4.92	52.0	7′-3″
B	61.25	5.71	75.8	8′-9″
C	130.00	4.62	130.0	11′-6″
D	71.25	5.75	88.7	9′-6″

(1) is the area required for total load computed by dividing the total load in kips by 8 ksf.

(2) is the dead load bearing pressure computed by dividing the dead load in kips by the area computed in step 1.

(3) is the new area required after adjustment computed by dividing the dead load in kips by the least bearing pressure computed in step 2.

(4) is the final specified size of the footing assuming 3-inch increments.

8

Piles, Piers, and Caissons

8-1. INTRODUCTION

It is recognized that spread footings are the quickest and most cost effective way to transfer the loads of a building into the ground. There are situations, however, in which spread footings cannot reasonably be used. An examination of the test boring log given in Figure 8-1 illustrates this point.

The general weakness and erratic nature of this soil precludes the use of spread footings. The only viable alternative is to use what is commonly called a deep foundation. This type of foundation consists of structural elements that extend far down into the ground to distribute their load through skin friction to a large mass of weak soil. They may also transfer load by direct bearing to the dense soil or bedrock, which must ultimately exist at some depth below ground level. Piles, piers, and caissons are the elements used in the construction of a deep foundation.

Load Capacity

The allowable load for which these elements may be designed is the lesser of the following two criteria:

1. The superimposed load from the building must be transferred to one or more of these elements. The load will be axial and the transfer will be in direct bearing. The capacity of the load that can be transferred to each element will be computed by multiplying an allowable stress in bearing times the area of contact. The allowable stress for each type of element is given in subsequent articles.

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

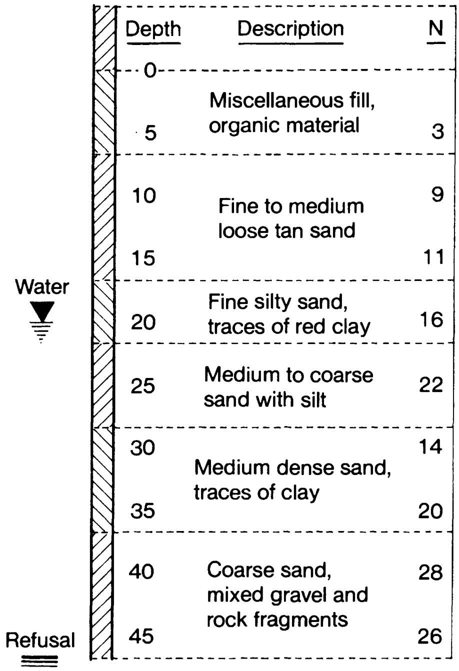


FIGURE 8-1. Sample log of test boring.

2. Depending on the type of element and on the type of soil, each element can develop a certain load capacity for transfer into the soil. This transfer will be made in one of the following ways:
   (a) The development of shear (friction) between the surface of the element and the adjacent soil, as given in Articles 8-10 and 8-11,
   (b) End bearing between the base of the element and the soil beneath, as given in Articles 8-12 and 8-13, or
   (c) A combination of shear (a) and end bearing (b).

   In each instance, the allowable load will be computed by multiplying the allowable stress by the surface on which the stress acts.

8-2. PILES

The term pile describes an element which is driven into the ground by heavy, hammerlike machinery. This process produces considerable noise and vibration. The pile may be timber, structural steel, steel pipe, or a steel shell filled with concrete after the shell has been driven. When a pile is driven into the ground, it must displace an equal volume of soil. The effect of this is to compact, or densify, the soil in the immediate vicinity of the pile. The soil pressure acting laterally on the pile, therefore, is increased. The friction developed between the surface of the pile and the surrounding soil is similarly increased.

Timber and steel shell piles usually transfer the bulk of their load through friction and for that reason they are frequently referred to as friction piles. Structural steel and steel pipe are piles that are more versatile, as they can be used not only to transfer load through friction to transfer a substantial amount of load through end bearing, particularly when bearing on rock.

Pile Installation

Piles are driven into the ground with a heavy hammer that strikes the top of the pile, thus forcing it into the ground. This is similar to the way in which a hammer is used to drive a nail into a piece of wood. In the case of piles, however, special pile driving equipment is required, as illustrated in Figure 8-2. There are several methods by which a pile can be driven. Two frequently used methods are single action driving and double action driving. In single action the hammer is raised

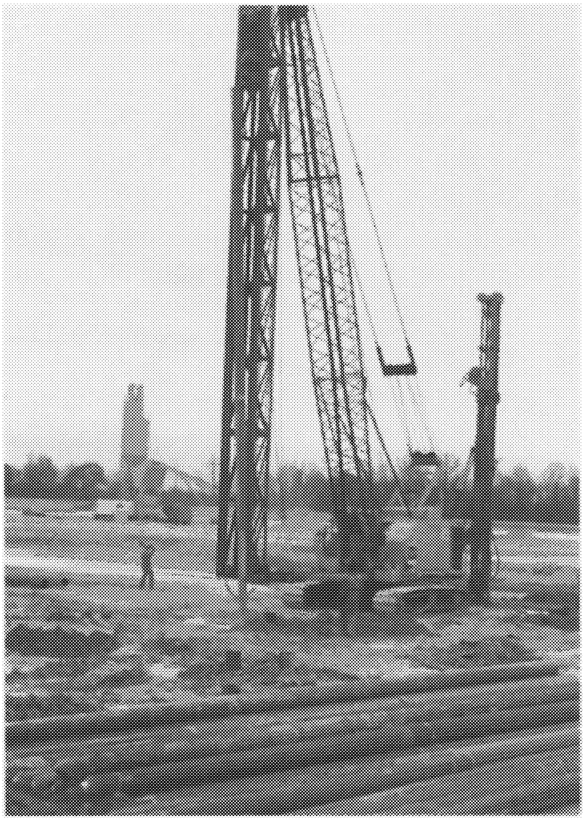


FIGURE 8-2. Installation of steel shell pile. [Ref. 15]

and then allowed to drop freely on to the pile. In double action the hammer is not only raised but is then accelerated downward, thereby increasing the striking force over that of free fall. In each case, the movement of the hammer is activated by steam or compressed air.

Pile driving is an operation that is inherently one of dynamic loading. It is the nature of pile driving, therefore, that the adjacent ground will be subjected to vibration. Depending on conditions, the intensity of this vibration can be substantial. There could be situations in which pile driving would be unacceptable because of the sensitivity of adjacent structures or other facilities to vibration. Pile driving is also very noisy. The use of piles in noise-sensitive areas, such as hospitals, may not be permitted.

Pile installation may be aided by one of the following methods:

1. *Predrilling:* When a pile must be driven through a layer of hard material, a pilot hole may first be drilled through that layer. The pilot hole must be smaller in diameter than the pile. Although this procedure makes the work of driving the pile easier, it also negates a considerable amount of the soil densification usually inherent to pile driving.
2. *Jetting:* This procedure is used to facilitate the installation of a pile in sandy soil. In this procedure a forceful stream of water is directed below the tip of the pile, thereby loosening and displacing the sand through which the pile must be driven. As in predrilling, there is a loss in soil densification and a corresponding loss in developed frictional resistance.

Pile Driving Formulas

Theoretically, it should be possible to mathematically express the correlation known to exist between the energy delivered to the pile by the striking force of the hammer and the energy absorbed by the penetration of the pile into the soil. Certainly it follows that the capacity of the pile should increase as the resistance to pile penetration increases. No pile driving formula, however, has yet been developed that will predict pile capacity accurately and consistently in all manners of soil. When used in conjunction with calculations and load tests formulas can, however, serve a useful purpose. One such purpose is to correlate the number of blows required to drive the pile a given distance, usually one inch, with that recorded for the test piles. For equal penetration the pile in question should develop a capacity equal to that of the test pile.

The Engineering News Formula that follows, has proved to be one of the more reliable pile driving formulas and is preferred because of its ease of use. The use of this formula, however, should be limited to driving solely in cohesionless soils. It should also be recognized that results computed by this or any other formula should be considered only as a preliminary guideline until correlated to field tests.

$$P = \frac{2W_hH}{S + 0.1} \quad \text{or} \quad \frac{2E}{S + 0.1} \tag{8-1}$$

In which:

P = the allowable pile load (pounds)
W_h = the weight of the striking part of the ram (pounds)
H = the height of free fall of the ram (feet)
W_hH = energy produced by a single acting hammer (ft pounds)
E = energy produced by a double acting hammer (ft pounds)
S = the pile penetration per blow after the pile has been driven to a depth at which successive blows produce approximately the same penetration (inches)

In pile driving terminology, the term hammer usually refers to all the moving parts. The term ram refers to the weight that strikes the pile. There are several different kinds of hammers. The two most frequently used in the market today are the single acting hammer and the double acting hammer. Single acting hammers use steam or compressed air to raise the ram, after which it is allowed to free fall onto the pile. Double acting hammers use steam or compressed air not only to raise the ram but also to accelerate it downward to the pile. For equal rams, therefore, double acting hammers produce more energy than single acting hammers because of their greater impact velocity. Note: Some pile driving companies use a differential steam hammer, the characteristics of which are similar to those of the double acting steam hammer.

It must be recognized that many intangibles can affect the accuracy of a pile driving formula. The reliability of soil information, the variation of soil throughout the site, the assignment of soil values, variation in equipment efficiency and the change in frictional resistance as a function of time are some of these intangibles.

Satisfactory pile performance combined with cost effectiveness is the ultimate goal of the foundation engineer. This can best be achieved by making decisions based on hands-on experience. An engineer inexperienced in pile construction should consider associating with an engineer who has had experience in that area. Pile driving companies are also an excellent source of information and will readily make their services available to the architect and engineer upon request.

Engineering Responsibilities

It is the responsibility of the project engineer to establish the estimated load bearing capacity of the piles during the design stage of the project. This will determine the number of piles required for the support of each load and will ultimately determine the total number of piles required for the project. This estimated number is the number of piles upon which the contractor will bid. It is also required that the project engineer establish the estimated length of each pile so that the contractor has a basis of length upon which to bid. During the actual driving operation certain piles may need to be driven deeper in order to develop their required strength while other piles could be shortened when the required strength can be developed at a lesser depth. This deviation in pile length between office and field is due to the variation in soil properties within the site and the many intangibles that are an integral part of substructure analysis.

The contractor, of course, needs a finite basis upon which to bid. This is why the engineer must establish the estimated number of piles and the estimated length of each pile. The contractor will then include in his base bid a price adjustment for the total number of piles added or deleted and the total number of feet of pile added or deleted. The add price is usually considerably more than the delete price. Therefore, it is in the owner's best interest that the engineer exercise careful judgement in his determination of these estimated quantities. The design work that the engineer must perform in determining pile capacities and lengths should be in accordance with Article 8-9.

The workmen in the field who are responsible for the pile driving operation have to know when to stop driving. Theoretically, this would be when the pile has been driven to the depth at which the specified strength is fully developed. The field has no way of knowing the actual depth at which this will occur, however, although they are guided, of course, by the estimated depths given on the engineer's drawings. The engineer must provide the field with a way of determining when to stop driving. The Engineering News Formula may be used for this purpose. The theory is that for a specified strength there is a specific number of hammer blows required to advance the pile one foot. Calculations based on this theory are illustrated in Examples 8-9 and 8-10.

The field is required to keep a running record of pile penetration correlated to blow count. This correlation may be given in the amount of penetration per blow or in the number of blows required for the pile to penetrate one foot. Because of the frequency with which the hammer strikes the field may prefer to measure the inches of penetration for a given number of blows. This measurement is then used to determine the equivalent penetration per blow. When this penetration is equal to, or less, than the value computed by the formula, it may be assumed that the pile has been driven to the depth required to develop the specified strength. Driving, therefore, can stop.

Battered Piles

Timber and steel shell piles can readily be battered when required. The usual reasons for specifying battered piles are as follows:

1. To support lateral loads in addition to vertical loads
2. To provide lateral stability

A typical battered pile installation is illustrated in Figure 8-3. Different arrangements of battered piles are shown in Figure 8-4, in which:

(a) indicates an arrangement that may be used to support lateral loads acting toward the battered pile, thereby placing that pile in compression
(b) indicates an arrangement that may be used to support lateral loads acting in either direction; it is also an excellent arrangement where substantial lateral bracing is required

FIGURE 8-3. Typical installation of battered piles. [Ref. 15]

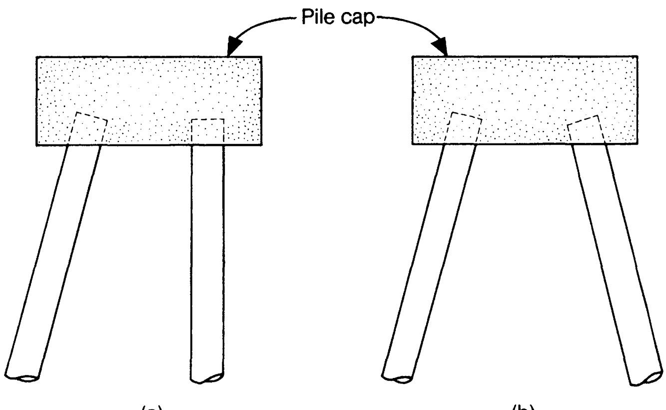


FIGURE 8-4. Battered pile arrangements.

8-3. TIMBER PILES

Timber piles are the least expensive of all piles and are still plentiful. Timber piles are made by stripping the bark off of relatively straight trees, removing limbs down to the trunk and permeating the member with creosote or some other preservative under pressure. The trees most frequently used for piles are pine, fir, and hemlock. The usual length of these piles is in the 25- to 35-foot range, although longer lengths are available on special order. Timber piles are particularly conducive to the development of skin friction due to their tapered shape. Resistance due to end bearing is of less importance in a timber pile because of the comparatively small bearing area. The allowable load on timber piles is usually in the order of 15 to 25 tons.

Because of the dynamics of driving, there is a tendency for the upper part of the pile to split or to be otherwise damaged. When this occurs it is necessary to cut off the damaged part of the pile. This may require an increase in the theoretical length of the pile. Damage of this type can be minimized by wrapping the upper 3 feet of the pile with heavy steel bands. In many pile installations this is routinely done.

The tip of a timber pile may also be damaged when driven through stiff soil or through weak or moderate soil containing loose stones or boulders. This type of damage can be prevented by attaching a metal driving point to the tip of the pile.

Load Transfer to Pile

Requirements relative to the physical characteristics, clear wood strengths and the determination of design stresses for timber piles are itemized in the following ASTM Standards:

ASTM Designation D-25: Standard Specification for Round Timber Piles

ASTM Designation D-2555: Test Methods for Establishing Clear Wood Strength Values

ASTM Designation D-2899: Test Method for Establishing Design Stresses for Round Timber Piles

The allowable working stress in compression parallel to the grain may be determined from the following formula, as specified in ASTM D-2899:

$$C = \frac{S - 1.645\ \mathrm{SD}}{1.88} = \mathrm{psi} \tag{8-2}$$

Where:

C is the working stress permitted in compression parallel to the grain for green, untreated piles,

S is the average crushing strength in compression parallel to the grain, as given in ASTM D-2555, and

SD is the standard deviation of the average crushing strength, as given in ASTM D-2555.

Note:

1. For sample values of S, SD, and C, refer to Table 8-1.
2. Multiply the working stress by the following factor, depending on the process of conditioning prior to treatment:

Air-dried	1.00
Steam conditioning	0.85

TABLE 8-1. Crushing Strength in Compression Parallel to the Grain for Timber Piles. [Ref. 2]

Species	*S* (psi)	*SD* (psi)	C^a (psi)
Douglas fir—coast	3784	734	1370
California red fir	2758	459	1065
Western hemlock	3364	615	1251
Longleaf southern pine	4321	707	1680

[a] Values are based on Formula (8-2).

3. For douglas fir and southern pine piles only, increase the working stress by $0.2L\%$, where L is the distance in feet from the tip of the pile to the critical section, as defined herein.

Timber piles subjected solely to axial compression shall be designed at a cross section through the pile called the critical section. This section is conservatively located as follows:

1. For purely friction piles the critical section may be taken at the butt, or at the top of the supporting stratum.
2. For piles driven to a hard stratum with sufficient energy to fully develop their ultimate bearing strength, the critical section may be taken at the tip.
3. For piles which transfer load through a combination of friction and partial end bearing, the critical section may be taken at the butt.

Load Transfer to Soil

The load on the pile will be transferred to the soil by a combination of shear and end bearing. In most instances calculations will show a much higher percentage of load being transferred through friction than through direct bearing. It is for this reason that timber piles are frequently referred to as friction piles.

There is also a question of the effectiveness of transfer in bearing when the pile is fitted with driving points. See Article 8-14.

8-4. STEEL SHELL PILES

Steel shell piles filled with concrete are a very popular mid-length, mid-capacity pile. Shell piles have 12-, 14-, 16-, and 18-inch butt diameters. The surface of the shell may be smooth or fluted and the shaft may be straight or tapered. A fluted, tapered pile, as illustrated in Figure 8-5, will develop the optimum in frictional resistance. A forged steel conical nose is factory-attached to the tip of each pile to facilitate driving. End bearing, therefore, may be diminished. See Article 8-14.

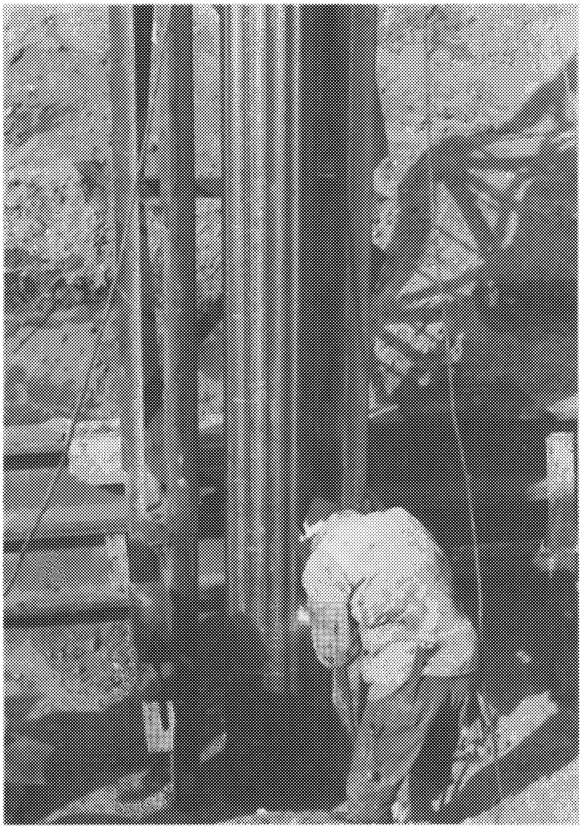


FIGURE 8-5. Close-up of a steel shell pile showing fluted sides. [Ref. 15]

When conditions require, the pile can be stiffened to prevent possible buckling of the shell while it is being driven. This is accomplished by installing a temporary support, called a mandrel, inside the pile. The mandrel is positioned to bear hard against the tip of the pile and, in order to facilitate driving, it must extend somewhat above the top of the pile. Since the mandrel exerts its driving force at the bottom of the pile, it is of interest to note that the mandrel actually is pulling the pile into the ground.

After the hollow shell has been driven to proper bearing, the mandrel is withdrawn, and the hollow shell is filled with concrete having a specified strength no less than 2500 psi. The concrete is never allowed to free-fall but must be installed by use of a tremie or an elephant trunk, as described in Article 8-20.

In bridge construction piles frequently project above the surface of the ground or water. These piles then become an important consideration in the architectural treatment of the entire area. An example of the aesthetic use of piles is shown in Figure 8-6.

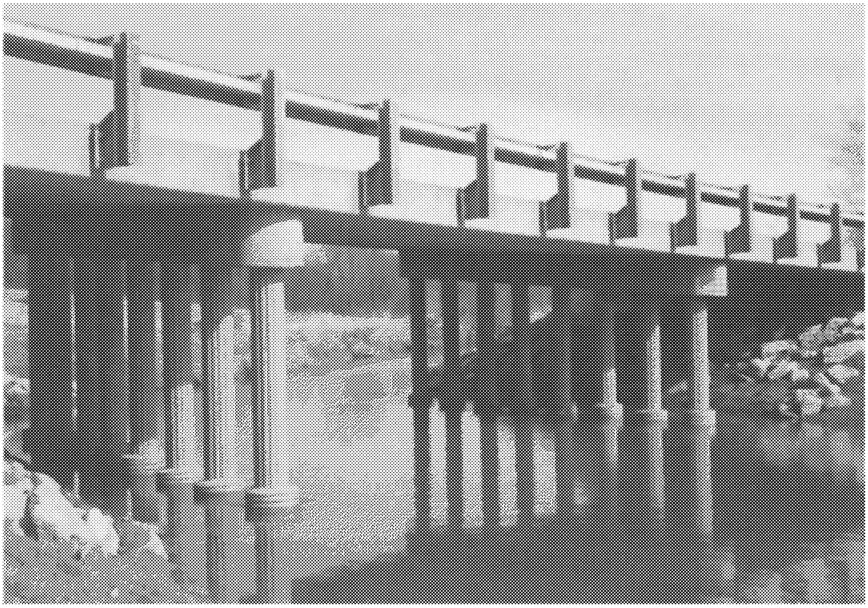


FIGURE 8-6. The aesthetic use of piles. [Ref. 15]

Load Transfer to Pile

The maximum load that can be sustained at the butt diameter by this type of pile is given by the following formula:

$$P_{\text{design}} = 0.35\ F_y A_s + 0.33\ f_c' A_c \qquad (8\text{-}3)$$

The physical properties needed for the application of this formula may be obtained from the manufacturer. The manufacturer will also have the results of a considerable number of load tests, and will make these available on request.

Formula (8-3) is actually an expedient and does not truly express the way in which the load is actually distributed between the two materials. The true distribution is based on the principle that the strain in the two materials must be equal, in which case:

$$\Delta L = \frac{\mathrm{PL}}{\mathrm{AE}}\ \text{steel} = \frac{PL}{AE}\ \text{concrete} \qquad (8\text{-}4)$$

As an example of the use of Formula (8-3) to determine pile capacity, assume a 14-inch, 9-gauge steel shell pile. The manufacturer has supplied the following data:

$F_y = 50$ ksi $\quad f_c' = 4$ ksi $\quad A_s = 6.75$ si $\quad A_c = 136$ si

Therefore, substituting into Formula (8-3):

$$P_{design} = 0.35 \times 50 \times 6.75 + 0.33 \times 4 \times 136 = 298 \text{ kips}$$

Load Transfer to Soil

The load on a steel shell pile will be transferred to the soil by a combination of shear and end bearing. Because of the use of a metal shoe to prevent damage to the tip of the pile while being driven, there may be a loss in end bearing effectiveness. Refer to Article 8-14.

8-5. STRUCTURAL STEEL HP PILES

The shapes used in steel pile construction are identified by the symbol HP and are available in 8-, 10-, 12-, and 14-inch sizes. These shapes are similar to *W* shapes, except that the web thickness has been increased to the same thickness of the flange in order to insure the integrity of the pile while it is being driven. Properties of these shapes are given in Part 1 of the AISC Manual of Steel Construction.

Structural steel piles are used to support major construction either by frictional resistance or by end bearing. The full advantage of this type of pile is realized, however, only when driven to bedrock; in which case the allowable load on each pile may exceed 200 tons. Structural steel piles are particularly suited for driving to end bearing, even though the penetration must be made through difficult stratas that contain hard lenses, boulders, or other obstructions. Because of the ease with which steel can be spliced by welding, a structural steel pile can be extended in length almost without limit.

HP shapes also serve as soldier beams and are frequently used in that capacity in projects involving major excavation. This use is briefly discussed in Article 10-4.

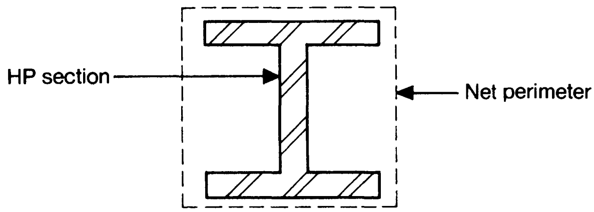
Load Transfer to Pile

HP piles are manufactured in steel having a minimum yield stress of 36 ksi, and a 50-ksi yield point is available for special conditions. The allowable working stress on these piles is usually limited by code to 0.35 F_y. The design load of the pile may be computed using the first term of Formula (8-3). An HP 12 × 53 pile, for example, with a cross-sectional area of 15.5 square inches would develop an allowable axial load of:

$$P_{design} = 0.35\ F_yA_s = 0.35 \times 36 \times 15.5 = 195 \text{ kips}$$

Load Transfer to Soil

HP piles should be designed to transfer their load through end bearing or through friction, but not both. HP piles can be designed for end bearing only when the



Section	Net perimeter
HP 14 X 14	4.75 feet
HP 12 X 12	4.00 feet
HP 10 X 10	3.33 feet
HP 8 X 8	2.67 feet

FIGURE 8-7. Net perimeter of HP steel piles for use in shear computations. [Ref. 24]

tip of the pile bears on hard rock. Otherwise, the pile should be designed as a friction pile. This is because the cross section of the pile can not effectively transfer load to soil through end bearing.

The question of the interaction between an end bearing pile and the rock upon which it is seated has been addressed by load tests in which the load was increased until there was evidence of failure. These tests gave clear evidence that primary failure occurred in the pile and not in the rock. The tests showed that the flanges or web of the pile will buckle locally at stresses approximately equal to the yield point of the steel. There will be no further penetration into the rock because rock confined laterally in its natural state can resist very high localized compression stresses without crushing. When designed as end bearing on rock, additional resistance based on shear is ignored.

When HP piles are installed as friction piles, the friction is assumed to be developed on the net perimeter of the cross section as given in Figure 8-7. When designed as friction piles, additional resistance based on end bearing is ignored.

8-6. STEEL PIPE PILES

Steel pipe is typically manufactured by the seamless or cold formed process, with yield points of 35 or 46 ksi. Properties of available pipe sizes are given in Part 1 of the AISC Manual of Steel Construction.

Steel pipe piles combine certain characteristics of HP piles and steel shell piles. Their driving characteristics are much like those of the HP piles, in that they can be driven through difficult stratas and can readily be extended to end bearing due to the ease of welding one section to another. As was the case with HP piles, the full advantage of the pipe pile is realized only when driven to bedrock.

The center of a steel pipe is open. It may be driven open or closed. When driven open the pipe will quickly fill with soil. Unless periodically removed, this soil will pack so tightly that the pile will drive as if it were closed. Soil removal is usually accomplished by compressed air or water jet. Soil can be prevented from entering the pipe by welding a steel plate to the end of the pipe. This will make the driving that much more difficult but avoids the problem of entrapped soil.

Load Transfer to Pile

A steel pipe pile shares with the steel shell pile the option of being filled with concrete. The maximum load capacity of the pile may be computed from Formula (8-3) for both the filled and unfilled conditions. For example, the maximum load capacity of an unfilled 8″ϕ @ 43.39 steel pipe pile having a cross sectional area of 12.8 square inches and a yield stress of 35 ksi would be computed using the first term of Formula (8-3):

$$P_{\text{design}} = 0.35\ F_yA_s = 0.35 \times 35 \times 12.8 = 156 \text{ kips}$$

If we assume that the pile is filled with 3000 psi concrete, then the additional load would be found using the second term of Formula (8-3):

$$P_{\text{design}} = 0.33\, f_c'A_c = 0.33 \times 3 \times \frac{\pi\ 7.625^2}{4} = 45 \text{ kips}$$

(where 7.625 inches is the inside diameter of the pipe.)

Load Transfer to Soil

The transfer of load from a steel pipe to rock or soil is the same as that for a HP pile with one exception. When a steel closure plate is welded to the pipe, the pipe can then be designed to transfer load directly to soil through end bearing alone, or in combination with friction.

8-7. CONCRETE PIERS

The term pier describes a type of foundation in which a continuous flight auger (much like a large corkscrew) is drilled into the ground, the earth spoils are removed from the hole and the resulting shaft is filled with concrete. An example of a pier hole being dug with a continuous flight auger is illustrated in Figure 8-8. Because piers are drilled, rather than driven, there is considerably less noise and vibration than in a pile installation. On the down side, however, piers develop less skin friction than piles due to the lack of soil densification inherent in the pile driving operation.

It is the nature of piers that their base is flat ended. They are, therefore, very effective in resisting load through end bearing. Modern drilling machinery permits

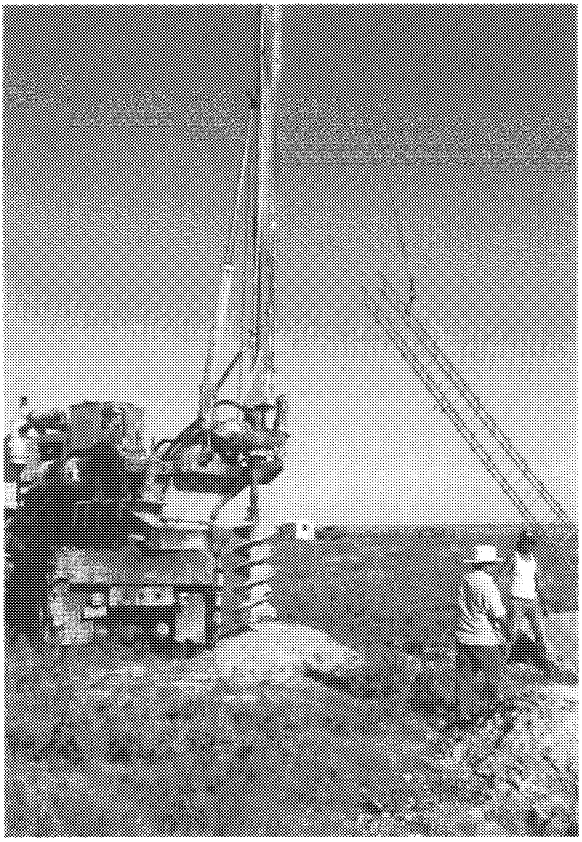


FIGURE 8-8. A pier being dug with a continuous flight auger.

a limited belling out at the base of the pier when in a cohesive soil. Such bells can not be inspected by hand, but can only be inspected from ground level.

There are several methods by which a pier can be installed, the selection of which depends on the characteristics of the soil through which the pier must be extended and on the preference of the contractor.

Installation in Clay

When the soil is predominantly clay, there will be sufficient cohesion so that the side walls of the open shaft will stand without collapsing or spalling off. This is the essential ingredient to the use of this method. Excavation is started by advancing the auger into the ground through a process of machine induced rotation. During this work the earth becomes enmeshed within the continuous cutting edge of the auger. The auger is advanced into the ground to a depth determined by the contractor, usually in the neighborhood of three to four feet.

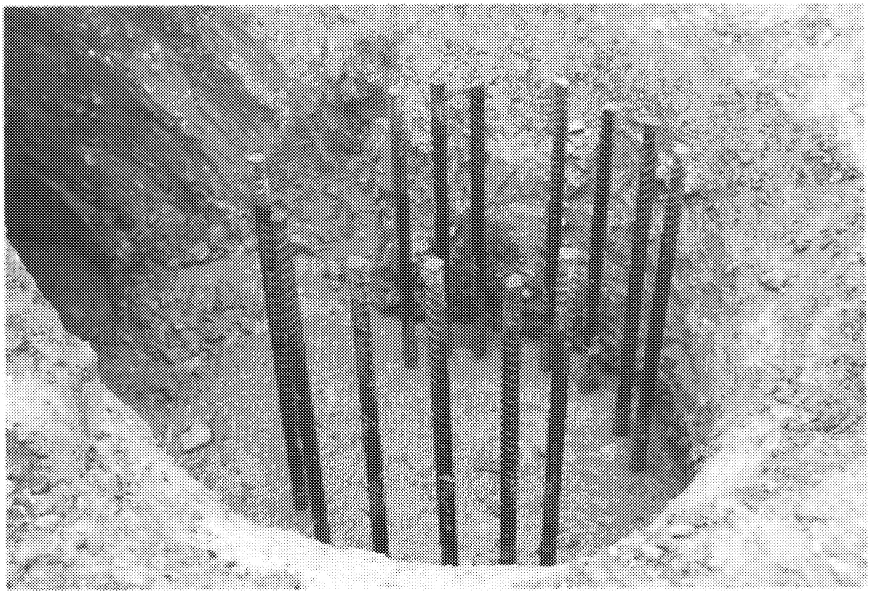


FIGURE 8-9. Placement of dowels in pier for anchorage of other elements.

Rotation is then stopped, and the auger is pulled up out of the ground, bringing the enmeshed earth spoils with it. These spoils are then removed by hand, and the auger is reinserted. This operation can be repeated as many times as necessary to produce a clean hole to the depth required by the contract drawings. After the hole has been completed a prefabricated reinforcing cage can be installed, if one is required. Most piers, however, are not reinforced except for the installation of dowels, which may be required to receive other elements of the building. A typical dowel installation is shown in Figure 8-9. The hole is then filled with concrete with the use of a tremie or an elephant trunk. Free fall must not be permitted, otherwise the materials will separate and the mass will lose strength. When using a tremie the concrete flows into the open hole only with the force of gravity. With the elephant trunk, the concrete must be pumped under a small head of pressure. In neither case will the concrete exert enough lateral pressure against the side walls of the hole to densify the soil significantly. The concrete will, however, flow into and fill any voids or large fissures extending out from the hole. The friction developed between the concrete and the soil in this type of installation will be less than that developed by any of the various piles installed by driving.

The general procedure by which a pier is installed in sound clay using a continuous flight auger is illustrated in Figure 8-10, in which:

(a) The auger is advanced and withdrawn in stages until reaching contract depth. During this work, the side walls must remain intact.

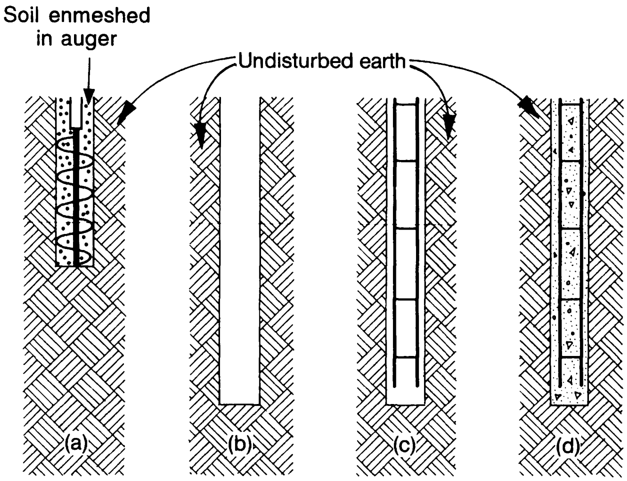


FIGURE 8-10. Procedure for installing a pier in clay using a continuous flight auger.

(b) The open hole is now at contract depth and is cleaned of any debris.
(c) Reinforcing, particularly dowels, can be placed at this time if required.
(d) The hole is concreted from the bottom up, using a tremie or elephant trunk.

Although this method of pier installation is the simplest of methods it is only applicable when the exposed earth walls remain stable throughout the process. The sporadic collapse of the walls due to weak pockets of soil or excessive seepage of water could render this method impractical. If it is demonstrated that this method is impractical, a bentonite slurry or a steel liner would have to be used to ensure the stability of the walls. The use of these materials is described in subsequent paragraphs.

Installation in Sand, With Slurry

When the soil is predominantly sand, there will be insufficient cohesion to keep the earth walls from collapsing into the hole. The earth spoils, therefore, can not be removed by simply pulling the auger out of the hole, as is done with an installation in clay. There are two methods by which this work can be done.

Method #1: This method of pier installation is similar to that described for clay except that a slurry of bentonite and water is circulated into the hole as it is being drilled. This slurry is used to prevent the cave-in which would normally occur in the side walls of a sandy or mixed-grained excavation. Bentonite, as discussed in Article 13-3, exhibits considerable increase in volume when exposed to water. When it is confined, as in the case of an open hole, the bentonite will exert a significant lateral pressure against the side walls of the hole. The slurry is continuously introduced into the open hole during the

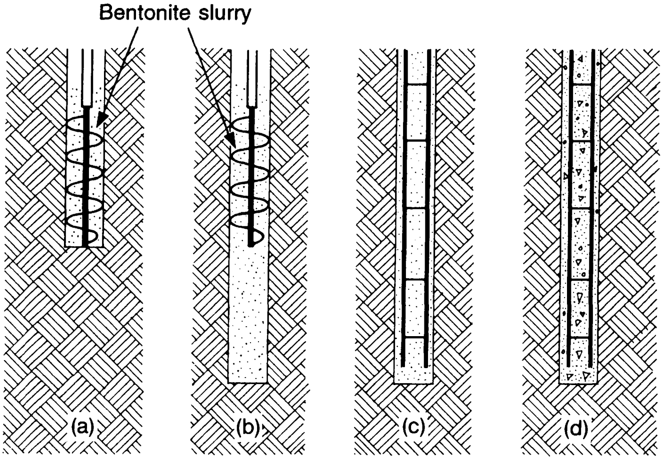


FIGURE 8-11. Procedure for installing a pier in sand using a continuous flight auger.

drilling process with a sufficient head of pressure to insure the integrity of the side walls against cave-in or influx of water. After the hole has been extended to contract depth and, while it is still filled with slurry, a reinforcing cage can be installed if one is required. The hole is then filled with concrete following the procedure described in the clay installation. Since the concrete is heavier than the bentonite slurry, it will force the slurry up to the surface from where it can be collected and disposed. Care must be taken in the disposal of the recovered slurry. Small volumes can probably be disposed of on-site. Larger volumes, however, should be taken to an approved off-site disposal area.

This method of installing a pier in sand is illustrated in Figure 8-11, in which:

(a) The auger is advanced and withdrawn in stages until reaching contract depth. During this time a slurry of bentonite is continuously added to the excavation.
(b) The auger, along with the last of the earth spoils, is withdrawn, leaving a hole cleaned of earth but filled with bentonite slurry.
(c) Reinforcing can be placed through the slurry at this time, if required. Dowels for the attachment of other building elements should be installed at this time.
(d) The hole is filled with concrete, using a tremie or elephant trunk extending through the slurry. The concrete displaces the slurry and forces it to the surface, where it is collected and disposed.

This method can also be used for pier installation in clay when the side walls lack sufficient cohesion to remain stable throughout the process.

Method #2: In this method, a temporary steel liner is lowered into the excavation after the excavation has been filled with the same bentonite slurry described

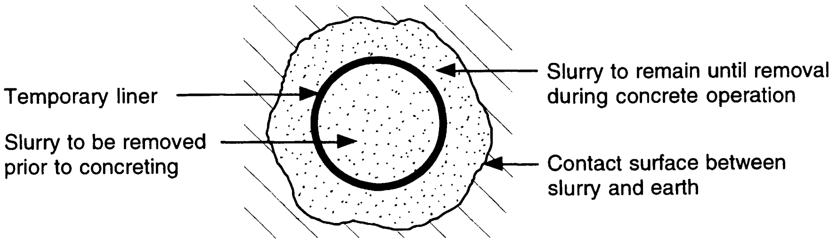


FIGURE 8-12. Temporary liners in place in a pier hole filled with slurry.

in Method #1. The liner can be installed during the drilling process or after the drilling has been completed. The diameter of the liner must be several inches less than the diameter of the drilled hole. The liner can be bedded against the bottom of the hole thereby producing a reasonable seal. Because the liner was lowered into the excavation through standing slurry there is now slurry within the liner and also in the space between the liner and the earth. This condition is illustrated in Figure 8-12.

The slurry within the liner is now removed. This results in an open hole which is relatively easy to reinforce and concrete. The temporary shell is withdrawn during the concreting operation. Care must be taken to maintain sufficient head of concrete within the liner in order to prevent reduction in the diameter of the pier due to earth pressure on the fresh concrete and to prevent extraneous material from contaminating the concrete. Note that during this process the concrete displaces the slurry that was contained in the space between the liner and the earth wall. This procedure is illustrated diagrammatically in Figure 8-13.

This method can also be used for pier installation in clay when the side walls lack sufficient cohesion to remain stable throughout the process.

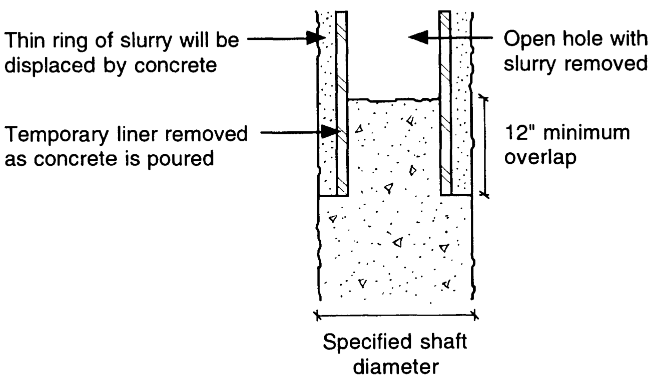


FIGURE 8-13. Procedure for removal of a steel liner during the pouring of concrete.

Installation in Sand, With Hollow Shaft Auger

A frequently used method by which a pier can be installed in sandy soil is to use a special type of auger having a hollow shaft. After the auger has been advanced to the intended depth, cement grout is pumped under considerable pressure into the hollow shaft by pumping equipment stationed at ground level. The grout travels down through the shaft and exits into the soil at the bottom of the hole. The force of the grout pushing against this soil lifts the auger with its enmeshed earth up and out of the hole. The lateral pressure exerted by the grout is sufficient not only to fill all voids and fissures but to actually compact and densify the surrounding soil as well. The volume of grout pumped into the hole is usually at least 15 percent greater than the computed volume of the open hole. This method of installation develops the maximum possible skin friction between a pier and the surrounding earth. One of the negative aspects of this method, however, is that there is no way in which the pier can be reinforced.

A typical pier installation using this method is illustrated in Figure 8-14, in which:

(a) The auger is advanced through the soil to contract depth.
(b) Grout is pumped through hollow shaft in the auger. This forces the auger and the enmeshed earth spoils to the surface. Note that as the auger is raised the grout pushes against the side walls, thus filling voids and fissures and densifying any soft spots.

Load Transfer to Pier

Concrete piers may be reinforced or unreinforced. The allowable load for which either type may be designed is given by Formula (8-3). When the pier is installed in cohesive soil or in sandy soil with a steel liner it is relatively easy to place

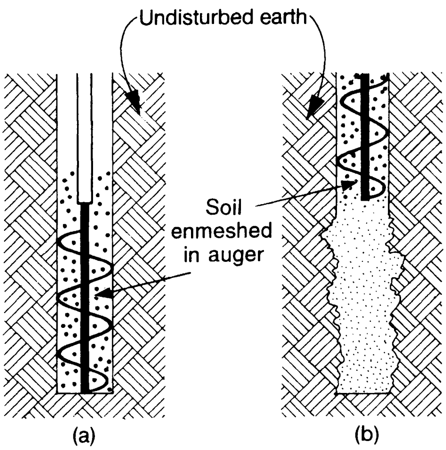


FIGURE 8-14. Procedure for installing a pier in sand using a hollow shaft auger.

reinforcing and to hold it in proper position. This is because the work is being done in an open hole. When the pier is installed in sandy soil with a bentonite slurry the reinforcing is more difficult to place since it must be pushed through the slurry.

Even though a pier may be unreinforced the load computed by Formula (8-3) is usually substantially more than the pier can transfer into the ground. The one exception to this is when the transfer is solely due to end bearing on rock or extremely hard material. In this case reinforcing may be required in order to fully develop the load transfer capacity of the pier.

Load Transfer to Soil

The load on a concrete pier is usually transferred to the soil through shear and end bearing. Note that end bearing developed by a pier is greater than that of a pile because of the difference in bearing area. When a pier bears on rock or extremely dense soil, the contribution of shear is ignored.

8-8. CAISSONS

The term caisson, as used in this text, is reserved for elements of large diameter, usually 24 to 48 inches, which are drilled into the ground much like a pier but which are then belled out at the base to provide additional bearing area. A typical caisson is shown diagrammatically in Figure 8-15.

The augers used for drilling the shaft of a caisson are substantial pieces of equipment, as illustrated in Figure 8-16. This auger is designed to be used for drilling in earth. Note in this figure that as the auger is withdrawn it brings to the surface the enmeshed earth spoils. Augers can also be equipped with carbide teeth when required for drilling in rock.

Caissons can be installed in any kind of soil. Installation in granular soils requires the use of a temporary steel liner that is lowered by machine into the shaft during the drilling process. Figure 8-17 illustrates such a liner being lowered into an open shaft. The side walls of cohesive soils may stand without the need for liners.

It must be noted that bells can only be dug in cohesive soils whose side walls will remain secure throughout the entire operation. In soils in which the side walls in the region of the bell will not stand or in soils in which the side walls are even suspect, bells cannot be used. In this event the shaft of the caisson must be increased to provide the required bearing area. This, of course, requires considerably more excavation, concrete and man hours.

When the soil is conducive to the installation of bells, the forming of the bells can be started by machine, but with large bells the work must ultimately be completed by hand. It is, therefore, a safety requirement that temporary steel liners must be installed in the shaft as the work progresses. This safety precaution,

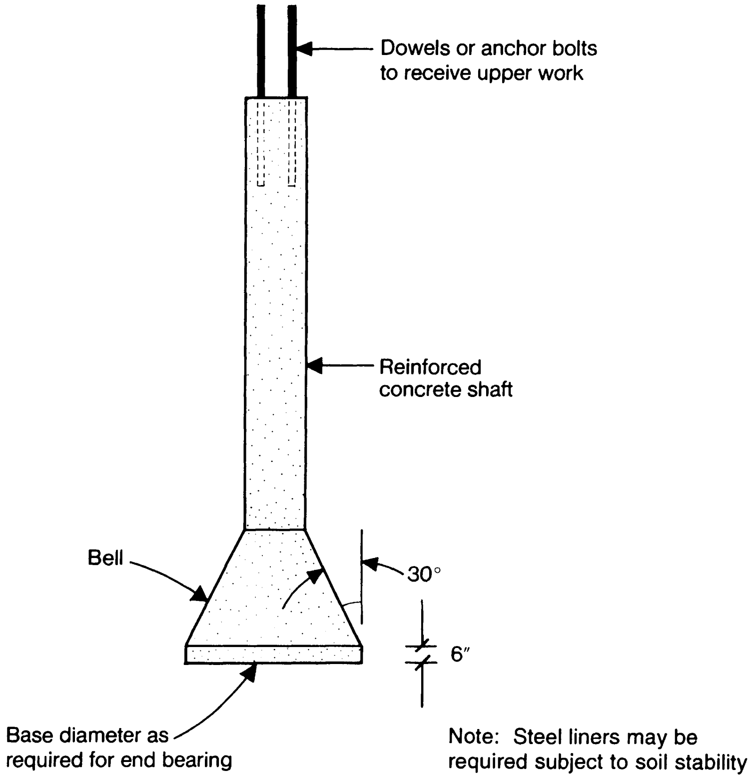


FIGURE 8-15. Typical caisson detail.

which cannot be waived, must be taken regardless of how stable the side walls of the hole appear to be.

Men working in a bell far below the surface of the earth may deplete more oxygen than can be replaced by the movement of air. It should be required, therefore, that a pump and hose be provided to insure a fresh and adequate supply of air at all times while men are in the bell area.

When there is ground water and a bell is to be formed, these liners can be welded so as to provide an essentially water tight excavation. Pumps must be provided, however, to take care of leakage or influx of water from the exposed earth forming the sides of the bell.

In the early days of bridge construction, there were situations where caissons had to be excavated to a considerable depth below standing water. The pumps of that day were frequently unable to control the leakage of water into the working area. In those situations, an air tight chamber would be built above the caisson. The workmen would wait in this chamber while compressed air was introduced into the area. The pressure of this compressed air would force the water out of the caisson, thus providing a reasonably dry area in which to work. Men working

FIGURE 8-16. A large diameter earth auger used in caisson construction.

in this area, however, could only work for short periods of time after which they had to return to the surface and to fresh air. Because of the crippling effect that compressed air can have on a man's blood when released too quickly, it was essential that the workmen go through a controlled process of decompression. This was accomplished by carefully monitoring the time spent in ascending up to the surface. If the ascent was too rapid, the man could suffer a life threatening disease called the bends.

During present day operations and before concreting, the bell must be cleaned out, leveled and inspected. Concreting is then performed with the use of a tremie or elephant trunk. The steel liner is usually lifted up and recovered as the concrete is placed. The lifting of the liner should always lag behind the depositing of concrete to insure the stability of the side walls. This detail was previously illustrated in Figure 8-13.

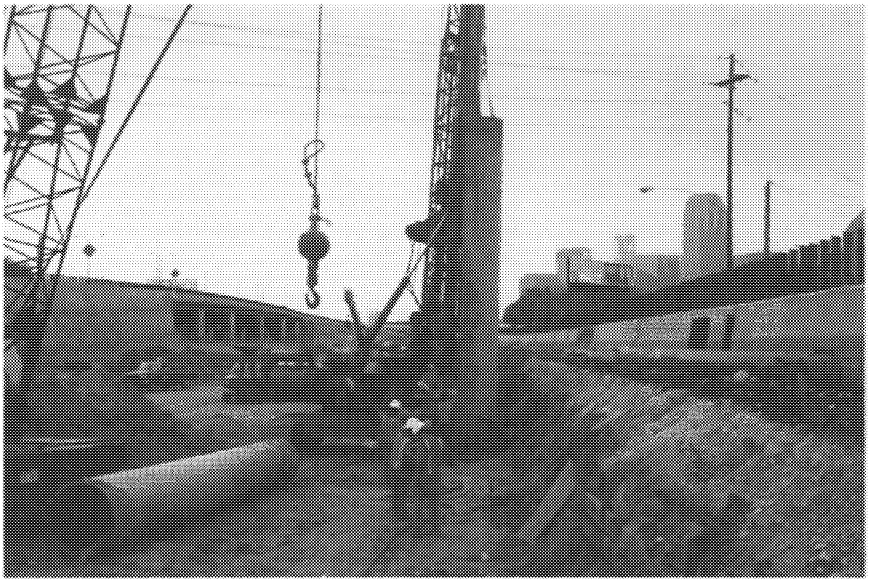


FIGURE 8-17. Installation of steel liners in pier and caisson work.

Load Transfer to Caisson

Caissons, like piers, may be reinforced or unreinforced. The majority of caissons are reinforced, however, because this will substantially reduce the diameter of the shaft, thus realizing a savings to both excavation and concrete. This is demonstrated in Example 8-8.

The allowable load that a caisson can support may be computed by Formula (8-3).

Load Transfer to Soil

The primary purpose of a caisson is to transfer extremely heavy building loads to bedrock or very dense soil through direct bearing. The majority of caissons, therefore, are belled out to increase the bearing area. Skin friction is also developed between the shaft of the caisson and the surrounding earth. This contribution, however, is small compared to that of direct bearing and is customarily ignored. Procedures for determining allowable bearing pressures are given in Article 14-7 for rock, Article 8-12 for clay, and Article 8-13 for sand.

8-9. ULTIMATE LOAD CARRYING CAPACITY

There are almost as many different ways used to compute the ultimate load carrying capacity of piles, piers, and caissons as there are books on the subject.

The procedures that follow draw on several of those different ways and on the experience of the author.

It is known that these elements derive their load carrying ability from shear or end bearing or a combination of both, as described as follows:

1. The development of shear between the surface of the shaft and the earth with which it is in contact. This shear is usually called skin friction when referring to cohesionless soils and may be called adhesion when referring to cohesive soils.
2. End bearing between the base of the element and the soil upon which it bears.

The ultimate load carrying capacity may be computed numerically by the following formula:

$$Q_{\text{ultimate}} = Q_{\text{shear}} + Q_{\text{bearing}} \tag{8-5}$$

Where:

Q_{shear} is numerically equal to the ultimate unit shear times the surface area of the shaft, and

Q_{bearing} is numerically equal to the ultimate unit bearing capacity times the bearing area of the base.

The concept of ultimate load resistance as presented in Formula 8-5 is illustrated graphically in Figure 8-18.

The determination of reasonable values for unit shear and unit bearing is a function of the type of soil through which the foundation is installed and upon which it bears.

The development of shearing resistance can only occur when there is very slight vertical slippage between the foundation and the adjacent soil. When the foundation has been extended to refusal or to a soil that is substantially stronger than the soil above, it is very doubtful that the slippage necessary to develop shear will occur. In this case, the foundation should be designed solely on the basis of end bearing.

8-10. ULTIMATE SHEAR STRESS DUE TO COHESION—CLAY

The ultimate capacity in shear developed by cohesion along the surface of contact between the shaft of a pile or pier and the surrounding clay may be computed from the following formula:

$$f = \lambda c \tag{8-6}$$

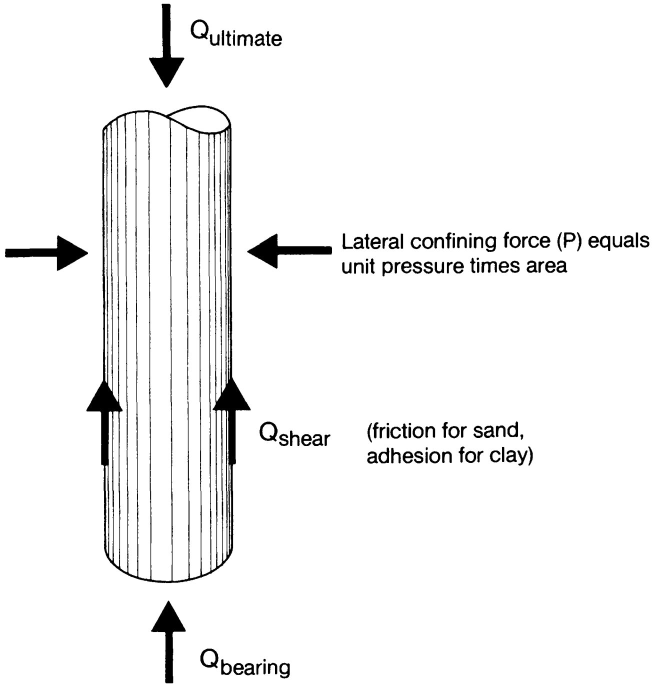


FIGURE 8-18. Factors contributing to the ultimate soil resistance of a pile, pier, or caisson.

In which:

f = the ultimate unit shear capacity (psf)
λ = a cohesion reduction factor
c = the unit cohesion, usually taken equal to one-half the unconfined compression strength q_u (psf)

The numerical value of this cohesion reduction factor is primarily dependent upon the following items:

1. The method by which the member is installed—remember, piles are driven, piers are drilled
2. For piles—the consistency of the clay, as given by its unconfined compression strength
3. For piers—was the bore hole dry or was a bentonite slurry added to control soil cave-in or influx of water

Pile driving remolds the clay in the immediate area of the pile. This has the effect of causing a temporary loss in cohesion for soft, sensitive clays. This is usually followed by a slow recovery. With stiffer clays, the loss is more permanent. Drilled shafts also exhibit a loss in cohesion due to the bleeding of water from the freshly poured concrete into the adjacent clay.

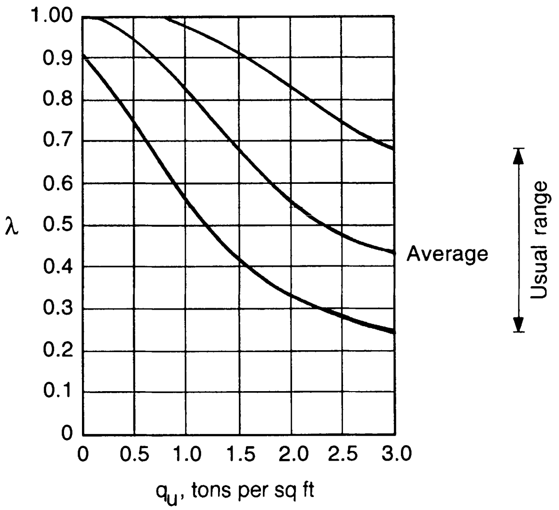


FIGURE 8-19. Cohesion reduction factor λ for piles installed in clay. [Ref. 16]

The cohesion reduction factors which follow reflect those that can normally be anticipated after recovery:

For HP piles: 1.0 for all clays
For other piles: Values are given in Figure 8-19
For piers, [Ref. 8]: a. 0.5 with a dry hole, but (f) shall not exceed 1800 psf
b. 0.3 with a bentonite slurry, but (f) shall not exceed 800 psf
For caissons: 0.0 because caissons are end bearing elements

In piers there is evidence of a zone of reduced contact at the top of the shaft where the shear strength due to cohesion may not be fully developed. This is due, in all probability, to the drying out of the upper strata of the clay and its subsequent shrinking away from the shaft. Within this zone it is considered prudent to ignore any effect of cohesion. Unless the height of this zone can be more accurately determined, a height of 5 feet is recommended.

8-11. ULTIMATE SHEAR STRESS DUE TO SKIN FRICTION—SAND

The concept of skin friction, as developed along the surface of contact between the shaft of a pile or pier and the surrounding sand, is illustrated graphically in Figure 8-20.

The ultimate capacity in shear thus developed can be computed from the following formula:

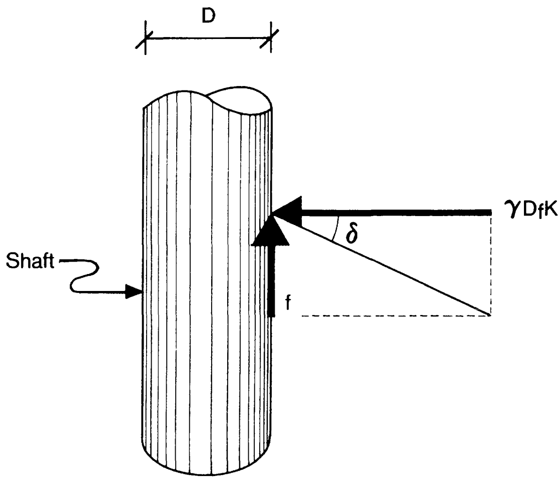


FIGURE 8-20. The development of skin friction in sand.

$$f = \gamma D_f K \operatorname{Tan}\delta \tag{8-7}$$

In which:

f = the ultimate unit shear capacity (psf)
γ = the unit weight of the in situ soil (pcf)
D_f = the depth below the surface of the earth at which the shear is to be calculated (feet)
γD_f = the effective overburden pressure (psf)—see note (a)
K = the coefficient of lateral pressure—see note (b)
$\gamma D_f K$ = the lateral pressure caused by the overburden pressure γD_f (psf)
$\operatorname{Tan}\delta$ = the coefficient of friction between the soil and the shaft, as evaluated in Table 8-2

a. It has been determined that the overburden pressure effective in producing skin friction between sand and piles increases in depth only to a certain depth of penetration. Below this depth, the effective pressure remains relatively

TABLE 8-2. Coefficient of Friction Between Pile Materials and Sand [Ref. 13]

Material	Tanδ
Concrete	0.45
Wood	0.4
Smooth steel	0.2
Rough, rusted steel	0.4
Corrugated steel	Use tan ϕ

constant. Tests and models indicate that the critical depth (D_c) ranges from about 10 pile diameters for loose sand to about 20 pile diameters for compact sand. Limitations on the critical depth D_c and the procedure by which the effective overburden pressure may be computed is given in Figure 8-21. This limitation applies only to piles. There is no such limitation for piers.

b. The numerical value of the lateral pressure coefficient K has been the source of much discussion but little agreement among those of the soils engineering community. There is agreement, however, that the value of K should be larger for piles (driven) than for piers (drilled). This is due to the densification caused by the driving operation. Conservative values for this coefficient are as follows:

For piles, [Ref. 18]:

K = 0.3 for silt
0.5 for loose sand
1.0 for dense sand
2.0 for pile clusters in dense sand

For piers, [Ref. 18]:

$K = 1 - \sin \phi$, where ϕ is the angle of internal friction

For caissons: $K = 0.0$ because these elements are considered to be end bearing

8-12. ULTIMATE END BEARING STRESS—CLAY

The ultimate capacity developed by end bearing on clay may be computed from the following formula, as adapted from Equation (5-3):

$$q_d = cN_c \tag{8-8}$$

In which:

q_d = the ultimate unit end bearing capacity (psf)
c = the minimum value of cohesion within a height of several feet above and below the foundation base (psf)
N_c = a bearing capacity factor, originally introduced in Article 5-3, and as modified herein for use with piles and piers.

TABLE 8-3. Summary of Values Used for End Bearing in Clay[a]

Consistency	Blow Count N^a	$q_u{}^b$ (tsf)	Cohesion[c] (psf)	N_c
Very soft	<2	<0.25	<250	—
Soft	2–4	0.25–0.50	250–500	6[d]
Medium	4–8	0.50–1.00	500–1000	7[e]
Stiff	8–15	1.00–2.00	1000–2000	8[e]
Very stiff	15–30	2.00–4.00	2000–4000	9[e]
Hard	>30	>4.00	>4000	10[d]

[a] Values from Table 3-2.
[b] Values from Table 2-3.
[c] Cohesion assumed equal to one-half of q_u.
[d] Values from [Ref. 13].
[e] Values interpolated by author.

Numerical values of c and N_c are given in Table 8-3.

Note: In adapting Equation (5-3) the coefficient of the first term was reduced from 1.2 to 1.0, the second term was deleted as being of minor effect, and the third term was deleted because N_γ is zero for a clay soil.

8-13. ULTIMATE END BEARING STRESS—SAND

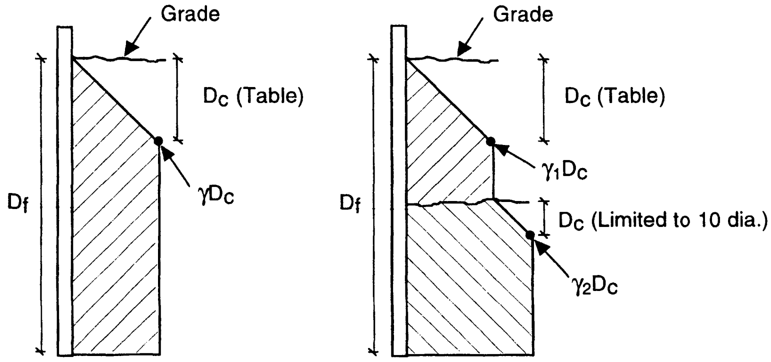
The ultimate capacity developed by end bearing on sand may be computed from the following formula, as adapted from Equation (5-3):

$$q_d = \gamma D_f N_q \tag{8-9}$$

In which:

q_d = the ultimate unit end bearing capacity (psf)
γ = the unit weight of the soil (pcf)
D_f = the distance below the surface of the earth to the depth at which the end bearing stress is to be calculated (feet)
γD_f = the effective overburden pressure whose value, in terms of γD_c, is limited to that given in Article 8-11 and Figure 8-21 (psf)
N_q = a bearing capacity factor originally introduced in Article 5-3

It is generally recognized that the numerical value of the bearing capacity factor N_q should be somewhat larger for deep foundations than for spread footings. Because of the wide variation in proposals, however, it is recommended that the values from Table 5-2 or Figure 5-2 be conservatively used for these calculations.



(a) Soil constant with depth

(b) Soil stronger with depth
$\gamma_2 > \gamma_1$

	Loose soil	Dense soil
ϕ	<30°	36-41°
D_c	10 dia.	20 dia.

(interpolate between)

FIGURE 8-21. Limitations to effective overburden pressure $\gamma\ D_f$ for piles driven into sand. [Ref. 11]

Note: In adapting Equation (5-3) the first term, involving N_c, was deleted because N_c is zero for a granular soil. The third term, involving N_γ, was also deleted because of its relatively minor contribution.

8-14. EVALUATION OF DESIGN BY FORMULA

General Considerations

Piles whose transfer of load is primarily one of frictional resistance or of frictional resistance combined with end bearing in earth should not be designed solely by formulas derived from theory. This category includes:

a. All timber and steel shell piles
b. Structural HP and steel pipe piles, unless they are driven to significant end bearing and designed solely as end bearing elements.

The reasons for this are as follows:

1. The technology from which the formulas were developed is not standardized. There is considerable diversity of opinions even among the experts.

2. The formulas assume that the soil within a given layer is homogeneous. Average values of the various properties can then be used. At best, this is an approximation.
3. Soils and soil properties may demonstrate a wide range of variation within a given site. What works for one area may not work for another.
4. The surrounding earth must have sufficient strength to hold the pile in proper alignment. Without this alignment the pile will fail. Visually dense soils can usually be assumed to provide adequate support. The adequacy of weaker soils must be suspect. Some authorities have indicated that the soil need only develop a cohesive strength of 100 pounds per square foot in order to adequately function as a lateral support element. Load tests resolve this problem completely.
5. Settlement may be a critical factor in the design. Foundations based on frictional resistance offer less assurance to the adequacy of settlement computations than do those that are primarily dependent on end bearing.
6. The judgment of the engineer who is responsible for the design of the pile is penalized because he cannot examine the supporting soil in its natural state.

The adequacy of the theoretical design for the previously noted foundations should be substantiated by on-site load tests, as described in Article 8-15.

The Effect of Driving Points

The tips of timber piles and steel shell piles are highly susceptible to damage while being driven. To prevent such damage these piles are usually fitted with steel shoes called driving points. These points are conical in shape but are rounded at the tip. Examination of recovered piles indicates the adequacy of this procedure in preventing damage to the pile tip.

However, the addition of these points raises the question as to the effectiveness of end bearing. These points are tapered—end bearing calculations are based on a level surface upon which to bear. Some engineers have expressed the opinion that load transfer in bearing should be ignored for those piles fitted with driving points.

The Effect of Tapered Surfaces

Timber piles and steel shell piles have tapered surfaces. The frictional resistance acting on these surfaces is produced by a confining force which is applied perpendicular to the surface. Because the surface is tapered a vertical component is produced by this confining force. This component adds to the total resistance of the pile. The same may be said of the tapered surface of the driving point. These additive effects are never considered in calculations of pile resistance. They may, however, give an indication as to why load tests frequently yield higher load capacities than calculations.

Bearing Capacity Factor N_q

This bearing capacity factor, as relating to shallow foundations, was introduced in Article 5-3. Values for this factor were given in Table 5-2 and Figure 5-2.

Investigations by Meyerhof, Berezontzev and others has resulted in the development of higher values for this factor for use in computing the end bearing capacity of a deep foundation.

As a comparison of the values between shallow and deep foundations assume an angle of internal friction of 36°, then:

a. $N_q = 37.8$ for a shallow foundation
b. N_q varies between 80 and 150 for a deep foundation, depending on the source

Although investigations indicate that the bearing factor for deep foundations should be substantially higher than for shallow foundations it can be seen that there is a diversity of opinion as to the actual value. When a pile is designed to transfer the load through a combination of frictional resistance and end bearing the accuracy of the N_q value is of less significance. For these reasons, and because of the need for a conservative approach to foundations, the author recommends that N_q be taken from Article 5-3 for all end bearing calculations.

8-15. LOAD TESTS FOR PILES

When piles are designed on the basis of friction or friction in combination with end bearing, the engineer is well aware of the many intangibles involved. This concern can best be alleviated by requiring that load tests be performed on the site of the proposed construction. Load tests, when conducted on full scale piles, are the only accurate way of determining the ultimate load capacity. Even though the requiring of load tests may be time consuming and costly, prudent engineering requires that this be done.

Load Test Scheduling

The architect, engineer and owner of the project must come to an understanding as to how this work is to be scheduled. There are two ways by which this work can be scheduled:

a. Preferably, this work should be scheduled to be performed as a separate contract during the design stage of the project. When load testing is performed during this stage, the engineer will have the timely benefit of the test results and can make any indicated changes in design before the project goes out for bids. This will also benefit prospective bidders because they can witness, and be privy to, information concerning the ease or difficulty with which the piles are installed. This is the preferred procedure and will ultimately result in an overall cost saving for the owner.

b. On the other hand, the owner may decide that it is in his best interest for the load tests to be included as part of the overall construction project. The project specifications must then include a procedure whereby modifications to the foundation design, if so indicated by the test results, can be made prior to the installation of any work that would be affected by such a change.

General Requirements

A minimum of two load tests should be performed on each site. Additional tests should be scheduled in any area where there is a marked change in the characteristics of the underlying soil. Each load test should be located close to one of the earlier test borings because of the known soil conditions at that location.

When piles are driven into clay the soil around the pile is effectively remolded. Clay that is remolded experiences a temporary loss of strength. The actual testing of these piles should not be started for at least two weeks after the pile has been driven. This allows the soil time to reconsolidate and regain its strength. When piles are driven into sand, the soil is densified, with a resulting increase in strength. The testing of piles, in this instance, need only be delayed for several days.

Testing Procedures

Load tests shall be performed in accordance with the following ASTM Standard:

> ASTM Designation D-1143: Method of Testing Piles Under Static Axial Compressive Load

There are several methods by which piles can be tested, as described in the ASTM reference. In each instance the underlying concept of the test is to determine settlement as a function of load.

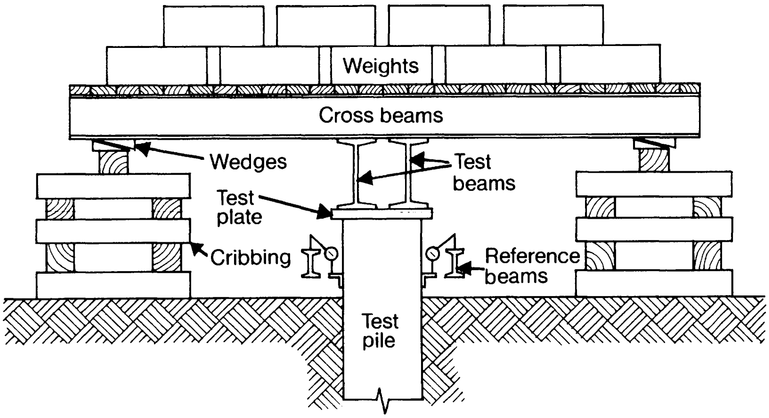
In what is called the Standard Loading Procedure, the pile is loaded to 200% of the design load, using 25% increments. Provided that the pile has not yet failed, the load is then removed, using 25% decrements. The vertical settlement or rebound of the pile is accurately monitored at all stages of loading. For the precise requirements relative to this procedure the reader is referred to the ASTM Test Method.

There are several methods by which the test piles can be loaded. One of the more frequently used methods is illustrated in Figure 8-22.

It should be noted that because of the need for accuracy in all readings, and because of the sensitivity of the equipment to changes in temperature, a sun screen must be constructed to protect the test sight from direct sunlight.

Test Evaluation

The purpose of a load test is to determine the ultimate strength of the pile under conditions simulating those to which it will be subjected in the building structure. The term "ultimate strength" as used here does not necessarily mean literal failure



(b) Using weighted platform

FIGURE 8-22. Schematic setup for a load test on a pile. [Ref. 2]

but is indicative of what might be called an upper limit of usefulness. Used in this sense, the ultimate strength Q_{ult} may be defined by any of the following criteria:

1. The load that produces a predetermined settlement, usually ½ to 1 inch
2. The load at which there is a disproportionate increase in the load-settlement curve, sometimes referred to as the break-in-the-curve method and normally requires the load-settlement curve to be plotted on logarithmic scale
3. Any other definition or limitation as specified in the governing building code

A typical load-settlement curve using arithmetical coordinates is illustrated in Figure 8-23. When using this curve the ultimate strength will be defined according to item #1.

A typical load-settlement curve using logarithmic coordinates is illustrated in Figure 8-24. Note that when plotted to a logarithmic scale the curve more closely

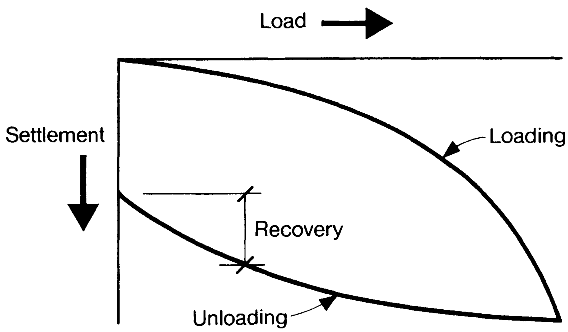


FIGURE 8-23. Load-settlement curve for a pile—arithmetic scale.

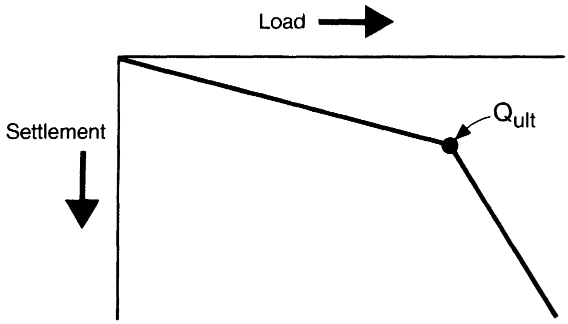


FIGURE 8-24. Load-settlement curve for a pile—logarithmic scale.

approximates two straight lines. When using this curve the ultimate strength will be defined as the load occurring at the break in the curve.

8-16. SAFETY FACTORS

The numerical value of a safety factor is generally determined by the extent of available information and the assurance with which the engineer can apply that information. Safety factors used in the design of deep foundations usually vary between 2 and 3, depending on circumstances. Factors which must be considered are the type of foundation, the variation in the soil as indicated by test borings, the properties of the soil as provided by laboratory analysis and the correlation of computed values with the results of load tests.

Safety factors should be higher when there are more intangibles in the design. Surely a foundation bearing on bedrock is a more positive thing than a foundation which develops its resistance solely through friction. Based on that premise the following safety factors are recommended for piles whose computed capacity will not be correlated by load tests:

1. Load transfer predominantly by end bearing—2
2. Load transfer by end bearing and friction in combination—2½
3. Load transfer predominantly by friction—3

When the computed capacity of a pile will be correlated by load tests it is the usual practice to use a safety factor of 2 for all load transfer conditions.

Load tests are normally not performed on piers. Safety factors, as previously listed for the three different conditions of load transfer, are applicable for all piers.

Caissons are predominantly end bearing foundations. A safety factor of 2 is customarily used.

8-17. PILE CLUSTERS

Load Capacity

When the load to be carried exceeds the design capacity of a single pile, then two or more piles must be used to carry the load. Such an arrangement may be called a group or a cluster. Because of the lateral pressure exerted by pile driving, the soil within the clustered area is highly compacted and acts integrally as a part of the cluster. The cluster, therefore, may be considered to be nothing more than a large, single pile.

Clusters may include any number of piles. A typical four pile cluster is illustrated in Figure 8-25. The center to center spacing between individual piles is normally established as the greater of three pile diameters or three feet. This dimension is fairly typical for piles and also for piers.

It is evident that for all clusters, the end bearing area of the cluster will be larger than the sum of the end bearing areas of the individual piles. From the end bearing standpoint, therefore, the ultimate strength of the cluster will exceed the combined strength of the individual piles.

In terms of frictional resistance, it can be demonstrated that the perimeter of the cluster is always larger than the perimeter of the individual piles.

Diameter of Pile	Perimeter of Single Piles		Perimeter of Cluster
D<1	4D	<	D + 12
D>1	4D	<	D + 12D

Since the perimeter of the cluster is larger than the sum of the perimeters of the individual piles, it follows that the frictional resistance of the cluster will exceed the combined frictional resistance of the individual piles. It is evident, therefore, that a pile cluster will have a capacity somewhat greater than the sum of the individual piles. There is, however, a lack of consensus among engineers

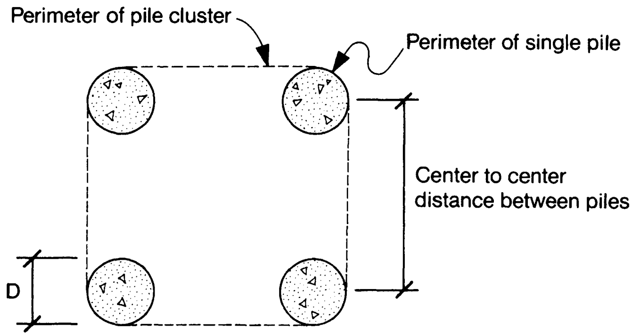


FIGURE 8-25. Typical arrangement of piles in a cluster.

as to exactly how to determine the frictional resistance of a pile cluster. For this reason the author recommends the following guideline:

> The frictional resistance of a pile cluster may be conservatively computed as the combined resistance of the individual piles.

8-18. ADVANTAGES AND DISADVANTAGES

Piles

1. Pile driving results in noise and vibration, either of which may preclude the use of piles in any given situation.
2. Piles require the use of poured in-place, concrete pile caps to transfer the load into the pile.
3. Steel piles are relatively easy to splice, permitting great depth. Steel piles, therefore, are frequently driven to bedrock and designed as end bearing piles.
4. Timber piles can be knocked off center by boulders. Steel piles, on the other hand, usually break through the boulder or displace it without misalignment.
5. Friction piles can be used to anchor a structure against the forces of uplift.
6. Piles can readily be battered. They can, therefore, be used as a positive means of providing lateral bracing, when required.
7. Piles driven into highly sensitive clays may liquify the soil, as described in Article 2-11. Liquification may also occur in saturated, loose sands. Liquification is a sudden occurrence in which the soil acts momentarily as a dense fluid, with a corresponding complete loss of shear strength. In extreme cases timber and steel shell piles have been known to actually float. It is evident that piles should never be used in soils susceptible to liquification.
8. Piles may wander off of dead center while being driven. The resultant eccentricity of load may induce sufficient instability to cause the pile to fail. There are three ways by which this problem can be avoided.
   (1) Brace all single piles with concrete struts extending in both directions.
   (2) Provide a two pile cluster and brace the cluster with concrete struts extending out from the weak axis of the pile cap. For a detail of this procedure refer to Figure 8-26.
   (3) Provide a three pile cluster, which is self bracing.

Piers

9. The augering operation used in the installation of piers produces little, if any, noise or vibration. Piers, therefore, can be used in situations where piles cannot be used.

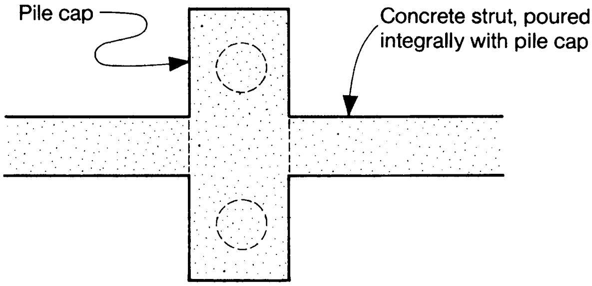


FIGURE 8-26. Typical method of pile cap bracing.

10. Piers usually have a larger diameter than piles. For a given length, therefore, piers will develop more frictional load carrying capacity.
11. Pier construction results in a certain volume of earth spoils, which must be used or disposed of properly.
12. Bad weather can adversely affect the installation of piers more so than piles, particularly due to the handling of earth spoils and concrete.
13. When drilled into clay, the shafts of piers may be visually inspected prior to concreting.
14. Piers, other than those drilled with a hollow shaft auger, can be reinforced if required by the loads. It is usually more expeditious, however, to increase the pier diameter thus avoiding the need for reinforcing.

Caissons

15. Caissons are essentially large piers and generally exhibit all of the advantages and disadvantages of piers.
16. Because caissons are large, they can be readily inspected. This eliminates a certain amount of guesswork, which is always present in the use of piles and piers.
17. Caissons constructed in cohesive soil can be belled out to provide additional bearing area. For this reason they can carry extremely heavy loads.
18. Caissons lend themselves to the installation of shaft reinforcing. This addition of steel will reduce the required shaft diameter. Since caissons are frequently of large diameter and height, a reduction in shaft diameter can result in a savings in both construction time and money.

Note that the problems caused by unintentional eccentricity as noted in item 8 do not occur as frequently in pier or caisson construction. Piles are driven, while piers and caissons are drilled. Drilling is the more accurate and easily controlled procedure. Even so, it may be considered prudent to laterally brace the top of a single pier or a two pier cluster. Caissons, on the other hand, have much more inherent stability because of their relatively large diameter.

8-19. APPROPRIATE USE OF PILES, PIERS, AND CAISSONS

It must first be understood that spread footings are invariably the proper foundation to use in the following instances:

1. When the soil within reasonable proximity to the surface of the ground can develop the required vertical and lateral resistance.
2. When the soil underlying the footing continues to increase in strength with depth.

For other conditions, as in the list that follows, the use of piles, piers or caissons may be appropriate.

3. When soil is very loose, as in the case of sand, or soft, as in the case of clay.
4. When the strength of the underlying soil does not increase with depth, or is erratic, with random layers of different soils and different characteristics.
5. When the strata upon which the footing would normally bear consists of expansive clay of too great a depth to reasonably remove and replace.
6. When very hard material, suitable for end bearing, is relatively close to the surface of the ground although lower than what would normally be customary for the use of spread footings.
7. When the site is covered by a thick layer of miscellaneous fill.
8. When there is no soil above bedrock which can develop the required bearing pressure. In this instance the architect should consider a change in program by providing additional basement and sub-basement areas. This change would extend the excavation down to bedrock and would permit the use of spread footings or shallow piers to rock. If it is decided not to make this change in program, foundations must then consist of piers or caissons drilled down to bedrock.
9. When soil densification is desirable piles can be very successfully used for this purpose. The vibration caused by the driving operation acts as a general densifier over a large area. The pile also displaces a certain volume of soil, thereby densifying the soil in the immediate area of the pile.

8-20. CONCRETING WITH TREMIE OR ELEPHANT TRUNK

Concrete is normally deposited into a shallow excavation as follows:

1. The mixed concrete is discharged from the delivery truck into an inclined metal chute whose free end is positioned over the point of intended deposit.
2. The concrete moves down along this chute by gravity and by occasionally being pushed along with hand tools.

3. After reaching the end of the chute the concrete free falls a short distance into the excavation.

There are two conditions when this method of concreting is not practical:

1. When the height of free fall as noted in item 3 exceeds 3 to 4 feet, thereby inviting segregation of materials—this occasionally occurs with spread footings but is a normal occurrence when concreting piers and caissons.
2. When the concrete must be deposited through a standing head of water.

For each of these conditions, proper construction techniques require that the concrete be deposited through a tremie or an elephant trunk. The tremie is a stationary pipe or tube which is fitted with a collection hopper at the top. Concrete is deposited into the hopper and then flows sluggishly down through the tremie into the area to be concreted. The elephant trunk, although similar to the tremie, uses a somewhat different principle. The elephant trunk consists of a flexible tube through which the concrete is pumped under pressure.

With either the tremie or the elephant trunk the height of free fall at the point of deposit can be adjusted as required to avoid segregation of materials.

When depositing concrete under a standing head of water it is very important to prevent the concrete from falling freely through the water, because it would mix with the water and disintegrate. On the other hand, when concrete is carefully deposited in still water without free fall, it will not absorb water nor will it separate. It is required, therefore, that the discharge end of the tremie or elephant trunk be kept beneath the surface of the freshly deposited concrete and that it be slowly withdrawn as more concrete is placed.

Both tremie and elephant trunk are widely used, and the choice is usually left to the personal preference of the contractor. A typical tremie operation is shown in Figure 8-27.

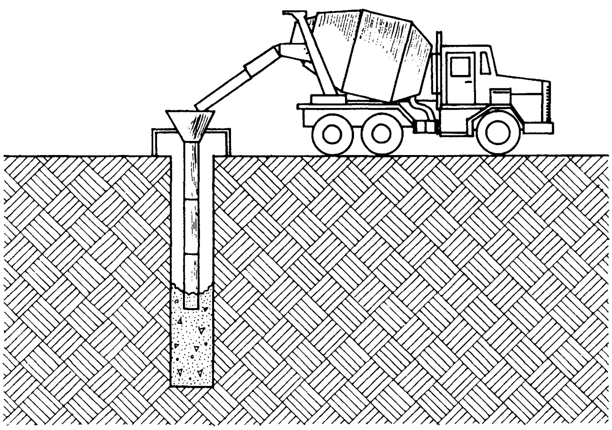


FIGURE 8-27. The use of a tremie in concreting.

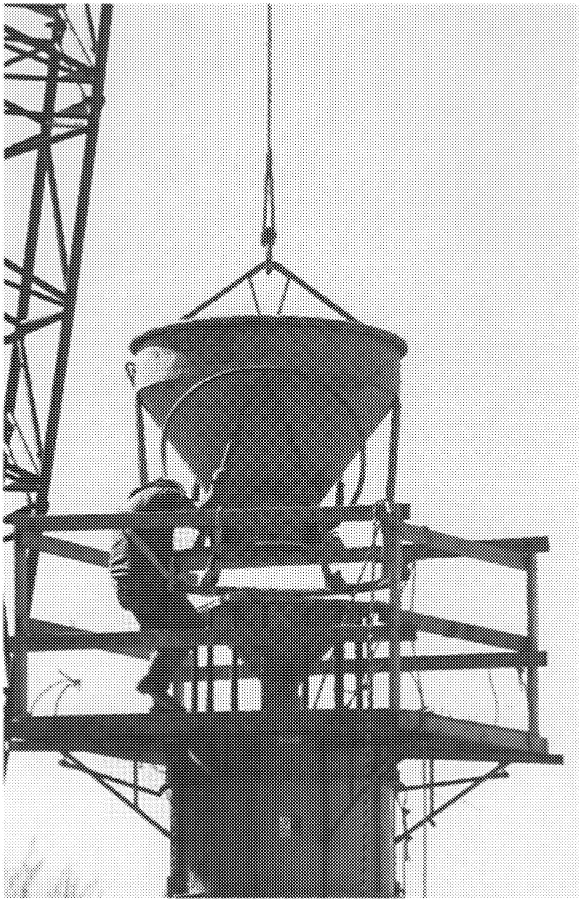


FIGURE 8-28. Concreting of piers with hopper and tremie.

Piers and caissons, when used in building construction, usually extend up to grade or to a floor for which the formwork has been placed. The top of the pier or caisson, therefore, is readily accessible for concreting since the concrete can be deposited directly from the truck or transported by buggy or conveyor belt.

The procedure for concreting piers in bridge construction differs from that used in buildings because the top of the pier usually extends a considerable distance above ground level or formwork. It is necessary, therefore, to deposit the concrete from the truck into a hopper which is then transported by crane to the pier. This is illustrated in Figure 8-28.

Because bridge piers are frequently used aethetically, as shown in Figure 8-29, special care must be given to the design and construction of formwork.

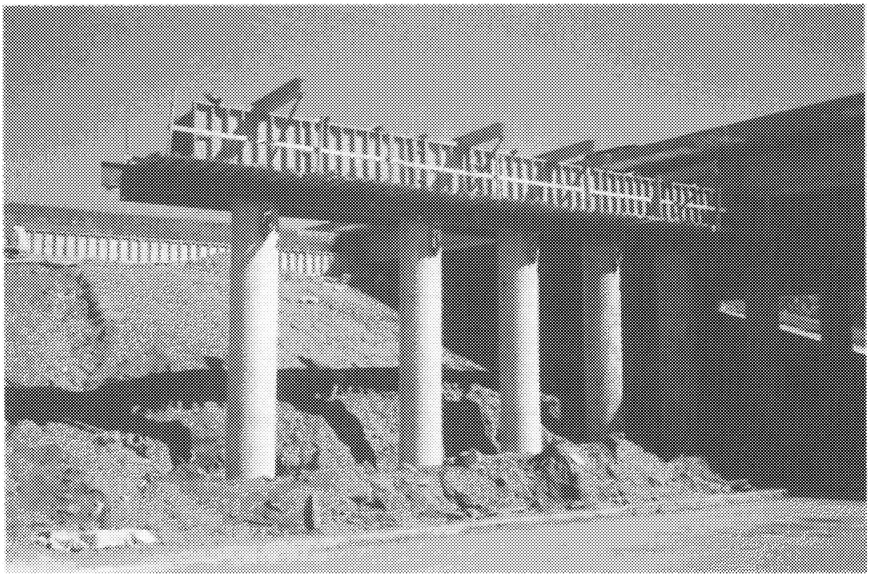


FIGURE 8-29. Exposed pier construction.

8-21. SAMPLE PROBLEMS

Example 8-1

Required: To determine the design strength of a 25-foot long, steam conditioned, longleaf southern pine timber pile driven into clay. Pile dimensions and soil characteristics are shown in Figure 8-30. The work shall be in accordance of Article 8-3.

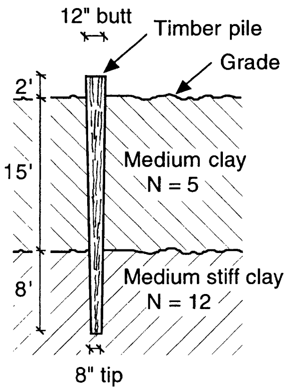


FIGURE 8-30. Example 8-1—Timber pile in clay.

The load carrying capacity of pile due to axial compression:

From Table 8-1: C = 1680 psi

Referring to the notes relating to Formula (8-2) the value of C shall be modified by conditioning (item 2) and by length (item 3). Taking the critical section of the pile at the butt:

$$C \text{ modified} = 1680\ [1 + 0.002 \times 23]\ 0.85 = 1{,}490 \text{ psi}$$

$$P_{\text{design}} = \frac{\pi\ 12^2}{4}\ [1490] = 174{,}000\# = 174 \text{ k} = 87 \text{ tons}$$

Note: This pile will be designed to transfer its load to the soil using a combination of shear and end bearing. The following constants have been interpolated from Table 8-3 and Figure 8-19:

Location	Consistency	N	q_u – tsf	c – psf	N_c	λ
Upper zone	Medium	5	0.625	625	—	0.92
Lower zone	Medium–stiff	12	1.570	1571	8	0.66

Also required are the following pile diameters:

$$\text{Average diam. in upper zone} = 4\left[\frac{15.5}{25}\right] + 8 = 10.5 \text{ inches} = 0.88 \text{ ft}$$

$$\text{Average diam. in lower zone} = 4\left[\frac{9.3}{25}\right] + 8 = 8.64 \text{ inches} = 0.72 \text{ ft}$$

Shear resistance developed by cohesion, from Formula (8-6):

Upper zone: $f = 0.92 \times 625 = 575$ psf
$P_{\text{shear}} = 575 \times \pi \times 0.88 \times 15 = 23{,}800\ \#$

Lower zone: $f = 0.66 \times 1571 = 1036$ psf
$P_{\text{shear}} = 1036 \times \pi \times 0.72 \times 8 = 18{,}700\ \#$

Total resistance due to shear = 42,500 #

Resistance developed by end bearing, from Formula (8-8):

$$q_d = 1571 \times 8 = 12{,}600 \text{ psf}$$

$$P_{\text{bearing}} = 12{,}600 \times \left[\frac{\pi\ 0.67^2}{4}\right] = 4{,}400\ \#$$

Total combined resistance, from Formula (8-5):

$$P_{\text{ultimate}} = 42{,}500 + 4{,}400 = 46{,}900\ \#$$
$$P_{\text{design}} = \frac{46900}{3} = 15{,}600\ \# = 8 \text{ tons}$$

A safety factor of 3 was selected for the following reasons, as stated in Article 8-16:

1. Load transfer is predominantly due to friction.
2. Load tests have not been specified.

Note: The allowable load transfer through shear and end bearing is considerably less than would be allowed due solely to axial compression on the pile. This is generally the case. Note also that end bearing adds little to the strength of this pile. This is why timber piles are frequently referred to as friction piles.

Example 8-2

Required: To determine the design strength of a structural steel HP pile driven into clay having the characteristics given in Figure 8-31. The pile is an HP 14 × 73, with an area of 21.4 square inches. The yield stress is 36 ksi. Load tests have been specified.

The load carrying capacity of the pile due to axial compression, from Formula (8-3):

$$P_{\text{design}} = 0.35 \times 36 \times 21.4 = 270 \text{ kips} = 135 \text{ tons}$$

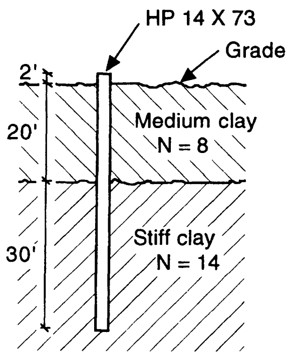


FIGURE 8-31. Example 8-2—Steel HP pile in clay.

Note: This pile will be designed to transfer its load to the soil through shear only. The following constants have been interpolated from Table 8-3. The cohesion reduction factor was given in Article 8-10.

Location	Consistency	N	c – psf	λ
Upper zone	Medium	8	1000	1.00
Lower zone	Stiff	14	1857	1.00

Shear resistance developed by cohesion, from Formula (8-6):

Upper zone: $f = 1.00 \times 1000 = 1000$ psf
$P_{\text{shear}} = 1000 \times 4.75 \times 20 = 95{,}000$ #

(The net perimeter or 4.75 inches is found in Figure 8-7.)

Lower zone: $f = 1.00 \times 1857 = 1857$ psf
$P_{\text{shear}} = 1857 \times 4.75 \times 30 = 264{,}000$ #

Total resistance due to shear:

$$P_{\text{ultimate}} = 95{,}000 + 264{,}000 = 359{,}000 \text{ \#}$$

Since load tests have been specified, the design load will be based on a safety factor of 2:

$$P_{\text{design}} = \frac{359000}{2} = 179{,}500 \text{ \#} = 89 \text{ tons}$$

In accordance with the reasons presented in Article 8-5 the possible development of resistance due to end bearing is not considered.

Example 8-3

Required: To determine the design strength of a 14″ diameter concrete pier drilled into clay soil of varying characteristics as shown in Figure 8-32. Installation was performed without the use of a bentonite slurry. Concrete is 3000 psi in ultimate strength.

The Load carrying capacity of an unreinforced pier due to axial compression, is calculated from Formula (8-3):

$$P_{\text{design}} = 0.33 \times 3000 \times \left[\frac{\pi\ 14^2}{4}\right] = 152{,}000 \text{ \#} = 76 \text{ tons}$$

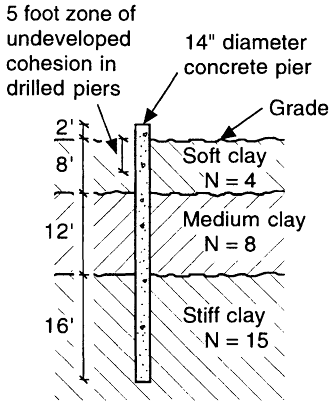


FIGURE 8-32. Example 8-3—Concrete pier in clay.

Note: This pier will be designed to transfer its load to the soil through a combination of shear and end bearing. The following constants have been taken from Table 8-3 and Article 8-10.

Location	Consistency	N	q_u – tsf	c – psf	λ	N_c
Upper zone	Soft	4	0.50	500	0.50	—
Middle zone	Medium	8	1.00	1000	0.50	—
Lower zone	Stiff	15	2.00	2000	0.50	8

The shear resistance developed by cohesion, from Formula (8-6):

Upper zone: $f = 0.50 \times 500 = 250$ psf
$P_{\text{shear}} = 250 \times \pi \times 1.17 \times (8 - 5) = 2{,}700$ #

(Note the reduction in effective contact height per Article 8-10)

Middle zone: $f = 0.5 \times 1000 = 500$ psf
$P_{\text{shear}} = 500 \times \pi \times 1.17 \times 12 = 22{,}000$ #

Lower zone: $f = 0.5 \times 2000 = 1000$ psf
$P_{\text{shear}} = 1000 \times \pi \times 1.17 \times 16 = 58{,}800$ #

Total resistance due to shear = 83,500 # = 42 tons

The resistance due to end bearing, from Formula (8-8):

$$q_d = 2000 \times 8 = 16{,}000 \text{ psf}$$

$$P_{\text{bearing}} = 16000 \times \left[\frac{\pi\ 1.17^2}{4}\right] = 17{,}200 = 8 \text{ tons}$$

The combined resistance, from Formula (8-5):

$$P_{\text{ultimate}} = 42 + 8 = 50 \text{ tons}$$

For piers whose load transfer is predominantly friction a safety factor of 3 is recommended:

$$P_{\text{design}} = \frac{50}{3} = 16 \text{ tons}$$

The load capacity of this unreinforced pier is 76 tons, but the transfer capacity to the soil was only 16 tons. This demonstrates why piers are generally do not require reinforcement. There are engineers, however, who prefer to add reinforcement in piers because of their length to thickness ratio. 0.5 to 1.0% reinforcing should be considered to be adequate.

Example 8-4

Required: To determine the design strength of a concrete filled steel pipe pile driven into a sandy soil having the characteristics shown in Figure 8-33. Load tests have been specified for these piles.

Given: Pile is a 10″ @ 54.74 steel pipe, F_y = 35 ksi, f'_c = 4000 psi, OD = 10.75 inches, ID = 9.75 inches, A_s = 16.1 square inches, A_c = 74.7 square inches.

The load carrying capacity due to axial compression, from Formula (8-3):

$$\begin{aligned} P_{\text{design}} &= 0.35 \times 35 \times 16.1 + 0.33 \times 4.0 \times 74.7 \\ &= 197 + 99 = 296 \text{ kips} = 148 \text{ tons} \end{aligned}$$

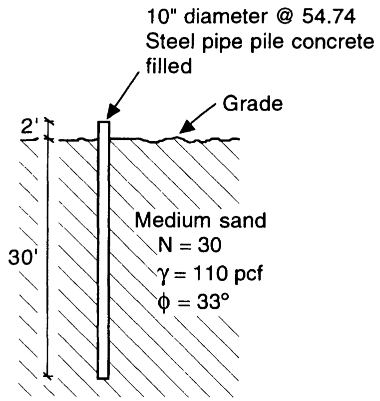


FIGURE 8-33. Example 8-4—Steel pipe pile in sand.

But according to Formula (8-4), the total load must be distributed between the steel and concrete in such proportion to insure that the shortening in each material will be equal. Therefore:

$$\Delta L = \frac{P_s}{16.1 \times 29.0} = \frac{P_c}{74.7 \times 3.64}$$

From which $P_s = 1.717\ P_c$.

If the maximum load that can be carried by the concrete is 99 kips, the load carried by the steel must be $1.717 \times 99 = 170$ kips.

This results in a revised $P_{design} = 170 + 99 = 269$ kips $= 134$ tons

The resolution of this discrepancy between code and performance is left to the reader.

Note: This pile will be designed to transfer its load to the soil through a combination of skin friction and end bearing. The following constants have been obtained from Article 8-11 and Table 8-2.

$K = 0.75$ (interpolated from [Ref. 18] for medium sand)
Tan $\delta = 0.4$ (assuming a rough, rusted surface)

Skin friction is a function of lateral pressure that, in turn, is a function of the effective overburden pressure. This is discussed in detail in Article 8-11. The depth D_c beyond which there is no appreciable increase in the effective vertical pressure is found to be 15 pile diameters, as interpolated from Figure 8-21(a), using $\phi = 33°$ as given. Therefore:

$$D_c = 15 \times 0.833 = 12.5 \text{ feet}$$

The maximum shear resistance developed by skin friction, from Formula (8-7):

$$f = 110 \times 12.5 \times 0.75 \times 0.4 = 412 \text{ psf}$$

The gradient of this shear along the length of the pile is shown in Figure 8-34, from which:

$$\begin{aligned} P_{shear} &= 0.5 \times 412 \times 12.5 \times \pi \times 0.833 + 412 \times 17.5 \times \pi \times 0.833 \\ &= 6{,}700 + 18{,}900 = 25{,}600\ \# \end{aligned}$$

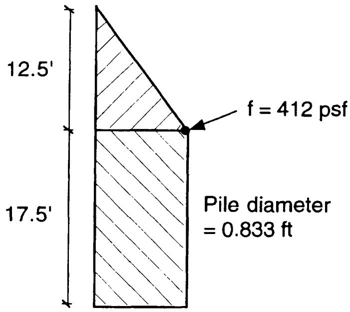


FIGURE 8-34. Example 8-4—Resultant shear gradient.

The resistance developed by end bearing, from Formula (8-9):

$$q_d = \gamma \, D_f N_q = 110 \times 12.5 \times 26.1 = 35{,}900 \text{ psf}$$

(D_c was limited to 12.5 feet. N_q was taken from Table 5-2.)

$$P_{\text{bearing}} = 35900 \times \left[\frac{\pi \; 10.75^2}{4 \times 144}\right] = 22{,}600 \text{ \#}$$

The total combined resistance, from Formula (8-5):

$$P_{\text{ultimate}} = 25{,}600 + 22{,}600 = 48{,}200 \text{ \#}$$

Load tests have been specified. A safety factor of 2 is recommended; therefore:

$$P_{\text{design}} = \frac{48200}{2} = 24{,}100 \text{ \#} = 12 \text{ tons}$$

Example 8-5

Required: To determine the design strength of a 25 foot long steel shell pile driven into sand having the characteristics shown in Figure 8-35. The pile has a yield stress of 36 ksi. Load tests have been specified for these piles.

Given: The pile has a butt diameter of 14 inches, a tip diameter of 8 inches and a wall thickness of 0.1793 inches. The pile has a steel area of 8.14 square inches at the butt, and 4.40 square inches at the tip. The shell will be filled with 3000 psi concrete after driving. The area of concrete at the butt is 136 square inches.

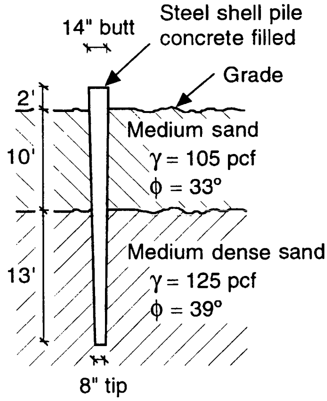


FIGURE 8-35. Example 8-5—Steel shell pile in sand.

The load carrying capacity of the pile due to axial compression, as determined from Formula (8-3):

$$P_{\text{design}} = 0.35 \times 36 \times 8.14 + 0.33 \times 3.0 \times 136 = 237 \text{ kips}$$

Note: This pile will be designed to transfer its load to the soil through a combination of shear and end bearing. The following constants have been taken from Article 8-11, Table 8-2, and Table 5-2:

Location	Description	γ	K^a	Tanδ	N_q
Upper zone	Medium	105	0.75	0.4	—
Lower zone	Med. dense	125	0.87	0.4	55.9

[a] Values were interpolated.

To compute the ultimate shear capacity of this pile, certain preliminary steps must be taken in order to determine the ultimate frictional stress as given by Formula (8-7).

The following pile diameters are computed:

Average diameter in the 10 foot upper zone = 1.03 feet
Average diameter in the 13 foot lower zone = 0.80 feet

The depth D_c beyond which there is no appreciable increase in effective vertical pressure is taken from Figure 8-21(b). Remember that in no case shall the thickness of the zone be exceeded.

Upper zone: D_c = 15 diameters, interpolated for medium sand, therefore:

$$D_c = 15 \times 1.03 = 10.3 > 10 \text{ foot zone, so use } 10 \text{ feet}$$

Lower zone: D_c = 10 diameters, therefore:

$$D_c = 10 \times 0.80 = 8.0 < 13 \text{ foot zone, so use 8.0 feet}$$

The shear resistance developed by skin friction is:

$$\textit{Upper zone } f = 105 \times 10 \times 0.75 \times 0.4 = 315 \text{ psf}$$
$$\textit{Lower zone } f = 125 \times 8 \times 0.87 \times 0.4 = 348 \text{ psf}$$

The gradient of this shear along the length of the pile is shown in Figure 8-36, from which:

$$P_{\text{shear}} = 0.5 \times 315 \times 10 \times \pi \times 1.03 + 0.5 \times 348 \times 8 \times \pi \times 0.80 + 315 \times 13 \times \pi \times 0.80 + 348 \times 5 \times \pi \times 0.80 = 25{,}200 \ \#$$

The resistance developed by end bearing, from Formula (8-9):

$$q_d = 125 \times 8 \times 55.9 = 55{,}900 \text{ psf}$$

(D_c in the lower zone was limited to 8 feet. N_q was taken from Table 5-2.)

$$P_{\text{bearing}} = 55900 \times \left[\frac{\pi\ 0.67^2}{4}\right] = 19{,}700 \ \#$$

The total combined resistance, from Formula (8-5):

$$P_{\text{ultimate}} = 25{,}200 + 19{,}700 = 44{,}900 \ \#$$

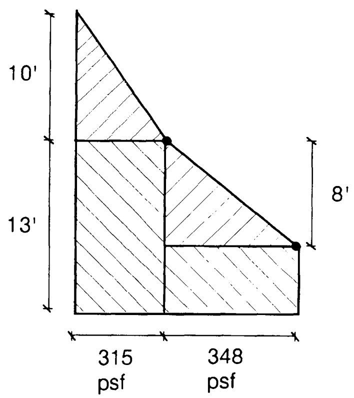


FIGURE 8-36. Example 8-5—Resultant shear gradient.

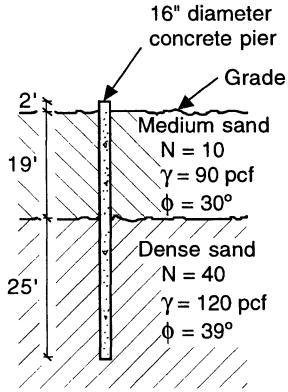


FIGURE 8-37. Example 8-6—Concrete pier in sand.

Load tests are specified; therefore, use a safety factor of 2. The design strength is:

$$P_{\text{design}} = \frac{44900}{2} = 22{,}450\ \# = 11\ \text{tons}$$

Example 8-6

Required: To determine the design strength of a 16″ diameter concrete pier drilled into a sandy soil having the characteristics given in Figure 8-37. Installation was by hollow shaft auger. Concrete will have an ultimate strength of 4000 psi.

The load carrying capacity of an unreinforced pile due to axial compression, is calculated from Formula (8-3):

$$P_{\text{design}} = 0.33 \times 4000 \times \left[\frac{\pi\ 16^2}{4}\right] = 265{,}000\ \# = 132\ \text{tons}$$

Note: This pier will transfer its load to the soil through a combination of shear and end bearing. The following constants have been taken from Article 8-11, Table 8-2, and Table 5-2:

Zone	K^a	Tan δ	N_q
Upper	0.50	0.45	—
Lower	0.37	0.45	55.9

[a]The value of K is taken as $1 - \sin\phi$.

The shear developed by skin friction is computed for each layer of soil according to Formula (8-7). Note that in the case of piers there is no limitation to the computed value of γD_f.

$$\textit{Upper zone } f = 90 \times 19 \times 0.50 \times 0.45 = 385 \text{ psf}$$
$$\textit{Lower zone } f = 120 \times 25 \times 0.37 \times 0.45 = 500 \text{ psf}$$

The gradient of this shear along the length of the pier is shown in Figure 8-38, from which:

$$P_{\text{shear}} = 0.5 \times 385 \times 19 \times \pi \times 1.33 + 0.5 \times 500 \times 25 \times \pi \times 1.33 + 385 \times 25 \times \pi \times 1.33 = 81{,}000 \text{ \#}$$

The ultimate resistance developed in end bearing is computed from Formula (8-9).

$$q_d = [90 \times 19 + 120 \times 25] \times 55.9 = 263{,}000 \text{ psf}$$
$$P_{\text{bearing}} = 263000 \times \left[\frac{\pi\ 1.33^2}{4}\right] = 365{,}400 \text{ \#}$$

The combined resistance, from Formula (8-5):

$$P_{\text{ultimate}} = 81{,}000 + 365{,}400 = 446{,}400 \text{ \#}$$

With load transfer by a combination of friction and end bearing, a safety factor of 2½ will be used:

$$P_{\text{design}} = \frac{446400}{2\frac{1}{2}} = 178{,}560 \text{ \#} = 89 \text{ tons}$$

Note that the design load due to transfer (89 tons) is considerably less than the axial load capacity (132 tons). This again demonstrates, as shown in Example

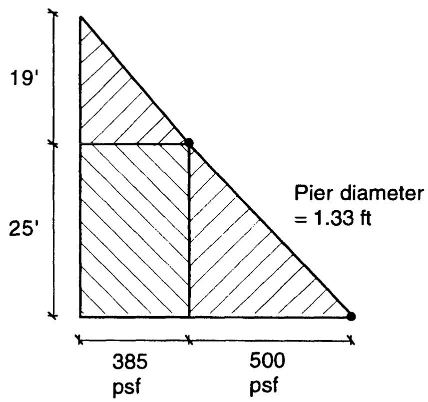


FIGURE 8-38. Example 8-6—Resultant shear gradient.

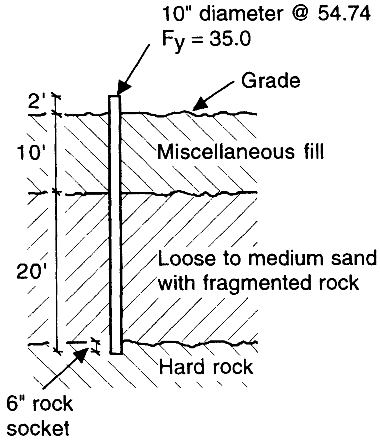


FIGURE 8-39. Example 8-7—Steel pipe pile bearing on rock.

8-3, that piers designed as shear and end bearing elements normally do not require reinforcing.

Example 8-7

Required: To determine the design strength of a steel pipe pile driven to rock through unsatisfactory bearing material as shown in Figure 8-39. Core boring samples indicate that the rock has a safe load bearing capacity of 40 tsf. The pipe is unfilled.

Given: A 10″ diameter steel pipe weighing 54.74 #/ft, having a steel area of 16.1 square inches and a yield stress is 35 ksi.

The load carrying capacity of pile due to axial compression, from Formula (8-3):

$$P_{\text{design}} = 0.35 \times 35.0 \times 16.1 = 197 \text{ kips} = 98 \text{ tons}$$

Because of the weakness of the overlaying soil, this pile will be designed to transfer its load to the sound rock through end bearing. The end of the pile will be fitted with a 10-3/4″ diameter bearing plate to insure full bearing on the pile diameter.

$$P_{\text{design}} = P_{\text{bearing}} = \left[\frac{\pi \ 10.750^2}{4 \times 144}\right] \times 40 = 25 \text{ tons}$$

Example 8-8

Required: To determine the shaft and bell diameters of a caisson which carries a superimposed load of 1800 kips. The caisson bears on bedrock having an allowable bearing pressure of 40 tsf. The depth to bedrock is 50 feet, as shown in Figure 8-40. Assume 3000 psi concrete, having a unit weight of 150 pcf.

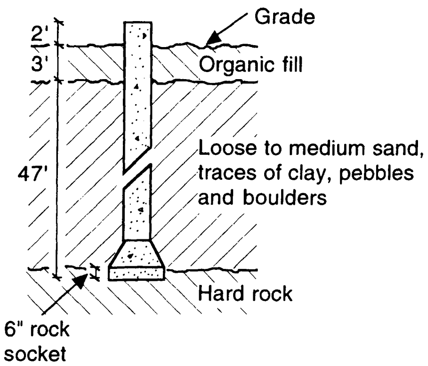


FIGURE 8-40. Example 8-8—Concrete caisson bearing on rock.

The material overlaying the rock is such that a full height steel casing will be required for all construction.

The required shaft diameter may be computed as follows:

$$\frac{\pi D^2}{4} = \frac{1800{,}000}{0.33 \times 3000} \text{ (unreinforced)}$$

A shaft diameter of 48.1 inches, therefore, is required. Prefabricated steel shafts are readily available in 6-inch increments. A 48-inch diameter shaft will be considered adequate.

If this shaft were reinforced its diameter could be reduced. Assuming an area of steel equal to 1.5% of the gross area, and assuming 60 ksi as the yield point of the steel, then:

$$0.35 \times 60 \times 0.015\ A_g + 0.33 \times 3 \times 0.985\ A_g = 1800$$

From which the gross area is 1395 square inches, and the diameter is 42″.

A comparison of the different shaft sizes yields the following:

	Diam. (in.)	Volume (cf)	Weight (kips)
Unreinforced	48	648	94
Reinforced	42	481	72

The design engineer will have to determine which of these two alternatives is the more cost effective and incorporate that method into his design.

Reinforcing, when specified, is fabricated in the shop to the required lengths and shipped to the job site bundled according to size, as shown in Figure 8-41. At the job site, the bars are sorted out and wired with circular ties into a cage

FIGURE 8-41. Example 8-8—Reinforcing grouped by size, ready for field fabrication into cages.

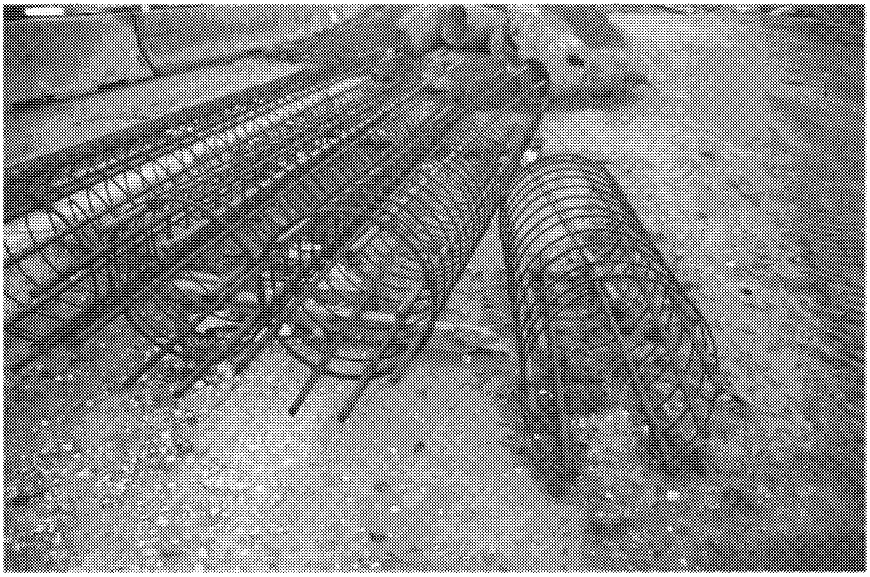


FIGURE 8-42. Example 8-8—Caisson cages ready for placement into forms.

having an outside tie diameter approximately 5 inches less than that of the caisson. Cages, ready for placement, are illustrated in Figure 8-42.

The required bell diameter, assuming an unreinforced shaft, is:

$$\frac{\pi D^2}{4} = \frac{1800 + 94}{40 \times 2}$$

The required diameter is 5.49 feet. Bells are usually specified in 6-inch increments, therefore, 5′-6″ bell should be used.

Modern methods provide for the excavating of bells by machine provided that the extension of the bell beyond the shaft is within the limits of the cutting blades. In this particular example it is highly unlikely that the soil in the area of the bell will have sufficient stability to permit the bell to be properly formed. If construction indicates that such is the case, there are two alternatives:

1. Increase the shaft diameter, thus negating the need for the bell. Due to the substantial increase in volume and dead load of concrete this will require a shaft somewhat larger than the original bell.
2. Use two or more smaller caissons to carry the superimposed load. These caissons, which are really piers, will have smaller shafts and no bells.

Example 8-9

Given: A Vulcan #1 single acting steam hammer, having the following characteristics:

Ram weight = 5000 # Stroke = 36″ Blows per minute = 60

Required:

First: To use the Engineering News Formula to determine the allowable load for which a pile can be designed, assuming that it required 16 blows to drive the pile one foot.

$$S = \frac{12}{16} = 0.75 \text{ inches per blow}$$

$$P = \frac{2W_hH}{S + 0.1} = \frac{2 \times 5000 \times 3}{0.75 + 0.10} = 35{,}300 \text{ pounds} \tag{8-1}$$

Second: To determine the required number of blows to drive the pile 1 foot if the pile is to be rated at 40,000 pounds.

$$40000 = \frac{2 \times 5000 \times 3}{S + 0.1}$$

From which $S = 0.65$ inches per blow

and the required number of blows per foot $= \dfrac{12}{0.65} = 19$ blows

Example 8-10

Given: A Vulcan #50C differential acting steam hammer, having the following characteristics:

Energy = 15,100 ft pounds   Blows per minute = 120

Required:

First: To use the Engineering News Formula to determine the allowable load for which a pile can be designed, assuming that it required 24 blows to drive the pile 1 foot.

$$S = \frac{12}{24} = 0.5 \text{ inches per blow}$$

$$P = \frac{2\,\mathrm{E}}{\mathrm{S} + 0.1} = \frac{2 \times 15100}{0.5 + 0.1} = 50{,}300 \text{ pounds} \qquad (8\text{-}1)$$

Second: To determine the required number of blows to drive the pile one foot if the pile is to be rated at 60,000 pounds.

$$60000 = \frac{2 \times 15100}{\mathrm{S} + 0.1}$$

From which $S = 0.4$ inches per foot

and the required number of blows per foot $= \frac{12}{0.4} = 30$ blows

9

Lateral Earth Pressure

9-1. GENERAL

The purpose of a retaining wall is to accommodate an abrupt change in grade as required by architectural or engineering considerations. In order to accomplish this, the retaining wall must restrain the lateral pressure exerted by the earth situated in back of the wall. A retaining wall cannot be properly designed until the designer has a clear understanding of the physical properties and in-place characteristics of the earth whose pressure the wall is to restrain. The purpose of this section is to develop the equations from which reasonable earth pressures can be computed.

9-2. THE CONCEPT OF LATERAL EARTH PRESSURE

The concept of lateral earth pressure can be explained by examining the behavior of a soil mass when it is free to move laterally and when it is restrained so that it cannot move laterally. A discussion of the following soil properties will provide insight as to this behavior.

1. The angle of repose, symbolized by (θ)
2. The angle of rupture, symbolized by (α)
3. The angle of internal friction, symbolized by (ϕ)

Angle of Repose

The angle of repose (θ), can best be explained by performing a relatively simple experiment in which a quantity of cohesionless soil is allowed to fall freely onto

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

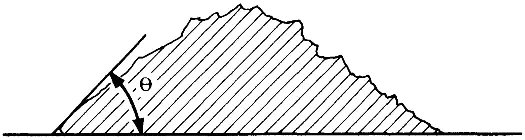


FIGURE 9-1. Profile through a freely deposited soil illustrating the angle of repose.

a level surface. When the soil is unrestrained laterally it will form a pile that is approximately conical in shape. The surface of the pile is actually somewhat crowned but since the amount is so small, it is customarily ignored. The angle that the surface of the pile makes with a horizontal line is called the angle of repose. The magnitude of this angle is a function of the size, shape and distribution of the individual soil grains and of the density of the mass. The concept of the angle of repose is illustrated in Figure 9-1.

As additional earth is deposited the pile will continue to grow vertically and horizontally, while maintaining the same angle of repose. Horizontal growth will only be prevented if the pile is laterally restrained by a physical barrier.

The formation of the earth pile is made possible by, and is clearly dependent upon, the frictional resistance developed between the individual grains of soil. Such a formation could not occur if this experiment were performed with marbles. Marbles, due to their round shape and smooth surface, do not have the ability to develop frictional resistance when laterally unrestrained. Soil grains, on the other hand, have this ability because of their irregular shape and surface texture. These physical characteristics cause the grains to physically interlock and to form a mass having a specific and measurable frictional resistance. The action illustrated in this experiment is one of shear, and all soils, to one degree or another, inherently possess the ability to develop a resistance to shear.

Coarse grained soils such as sand and gravel develop their resistance to shear through friction. Fine grained soils such as silt and clay develop their resistance through a property called cohesion. This property is described in Articles 2-13, 4-1, and 4-2.

Angle of Rupture

When a mass of granular soil is laterally restrained and the restraint is suddenly removed, a roughly triangular wedge of earth will slide downward and outward toward the released side. The plane upon which the wedge will slide is called the plane of rupture and the angle which this plane makes with the vertical is called the angle of rupture (α). This plane is actually not flat but is slightly concave. The amount of curvature is so small, however, that, as in the case of the angle of repose, it is customarily ignored. The sliding action of this earth wedge is demonstrated in Figure 9-2, in which a mass of dry sand and gravel is contained within a box having one side constructed as a removable panel.

When the removable panel is lifted the upper wedge of earth will immediately

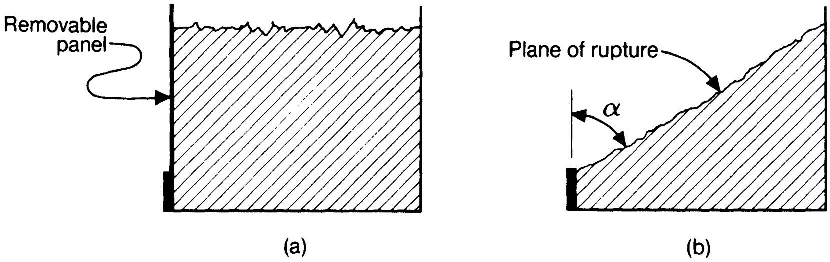


FIGURE 9-2. Action of an earth mass when free to slide. (a) before release, (b) after release.

slide down and out of the box. The movement of this earth wedge upon the release of physical restraint is clear evidence of the existance of lateral earth pressure.

It should be noted that this test was performed on dry granular soil. This kind of soil responds very quickly to the release of restraint and is, therefore, a good material with which to illustrate this point. All other soils will ultimately respond in the same general way. This is true for all soils, granular and cohesive, regardless of their characteristics or physical properties.

Angle of Internal Friction

The angle of internal friction (ϕ), as first discussed in Article 2-12, plays a significant role in the theory upon which the development of lateral earth pressure is based.

Representative angles for various general soil classifications are given in Table 2-4. A possible source of error exists in the use of that table because of the user's interpretation of the description of the soil. When using information obtained without benefit of tests, good engineering judgement dictates the selection of conservative values. The following rule of thumb may serve as a guideline in that selection:

> The angle of internal friction depends, to a large extent, on grain size. Therefore, the magnitude of this angle can be assumed to vary between the following general soil classifications as follows:

Gravel > sand > silt > silty clay

It has been noted that for a coarse grained soil, the angle of internal friction, and the blow count N, as recorded during the standard penetration test, are both related to the relative density of the soil. An approximate correlation exists, therefore, between these two properties. This correlation has been illustrated in Figure 3-9.

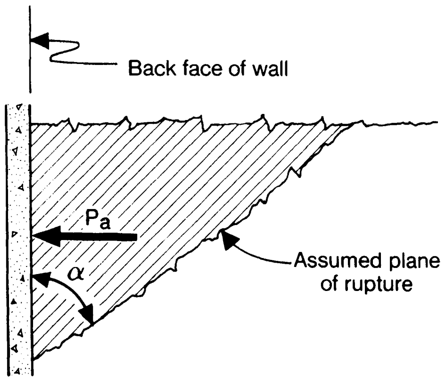


FIGURE 9-3. The development of lateral pressure due to a tendency for a wedge of soil to slide down along a potential plane of rupture.

9-3. ACTIVE EARTH PRESSURE AND THE PLANE OF RUPTURE

It has been previously demonstrated that when soil is deposited against a wall it will exert a lateral pressure against the wall. This pressure is called active pressure and is symbolized by (P_a).

Soil deposited against a wall is called backfill. As each successive layer of backfill is deposited, it will compact and densify the layers beneath. Compaction of these layers will alter the soil characteristics in regards to the angle of repose, angle of rupture, and angle of internal friction. This will ultimately affect the magnitude of the active pressure exerted by the soil mass against the wall.

The soil mass producing this active pressure is shaped like a wedge, whose profile is illustrated in Figure 9-3.

The tendency of the earth wedge to slide laterally is resisted by (a) the wall, and (b) the internal shearing resistance of the soil. On one particular plane, called the plane of rupture, the shear induced by the weight of the earth wedge will overcome the shearing resistance developed along that plane, thereby nullifying any resistance attributed to the soil. All of the lateral pressure caused by the earth wedge must then be resisted solely by the wall. The magnitude of this pressure can be found by applying the principles of static equilibrium to the forces acting on the earth wedge.

9-4. THE WEDGE THEORY OF ACTIVE EARTH PRESSURE

The wedge theory of active earth pressure developed in this section is based upon the following assumptions. The first three of these are somewhat conservative in theory and, therefore, on the side of safety. The remaining assumptions are for the purpose of presenting the computations in the simplest form and will be modified later in the chapter.

1. The earth behind the wall is free of fines and has no measurable cohesion. Therefore, in accordance with the Coulomb equation for shear strength:

$$c = 0 \qquad \text{and} \qquad s = p \tan \phi$$

2. The stabilizing effect of the earth in front of the wall is ignored in all computations.
3. The back face of the wall is vertical, and no friction or cohesion is developed between the backfill and this surface of the wall.
4. The surface of the earth in back of the wall is level.
5. There is no surcharge.
6. There is no standing ground water behind the wall, and there is a positive drainage system that will prevent any build up of water pressure behind the wall during extensive rainfall.
7. The earth behind the wall is reasonably uniform, and the angle of internal friction is reasonably constant throughout the mass.

In this analysis the earth wedge is isolated as a free body diagram, on which all of the external forces acting upon it are indicated. Equilibrium equations are then developed for the components of the forces acting vertically and horizontally on the wedge. The end result of this analysis will be the determination of lateral pressure (P_a) for which the wall must be designed. The free body diagram of the wedge is shown in Figure 9-4, in which:

$\alpha =$ the angle of rupture, defining the assumed plane of rupture
W = $0.5\ \gamma h\ [h \tan \alpha]$ = the weight of soil comprising the earth wedge
$P_a =$ the lateral force exerted on the wall by the sliding action of the earth wedge—this is the force that is the main thrust of this investigation
$R =$ the horizontal reaction that the wall must provide in order to produce equilibrium—this reaction is equal and opposite to P_a
$p =$ the confining pressure acting normal to the plane of failure
$s =$ the shear stress acting along the plane of rupture—numerically equal to $p \tan \phi$

$p \left[\dfrac{h}{\cos \alpha}\right]$ = the confining force acting normal to the plane of rupture

$s \left[\dfrac{h}{\cos \alpha}\right] = \dfrac{[p \tan \phi]\ h}{\cos \alpha}$ = the shear force acting along the plane of rupture to resist the sliding action of the earth wedge

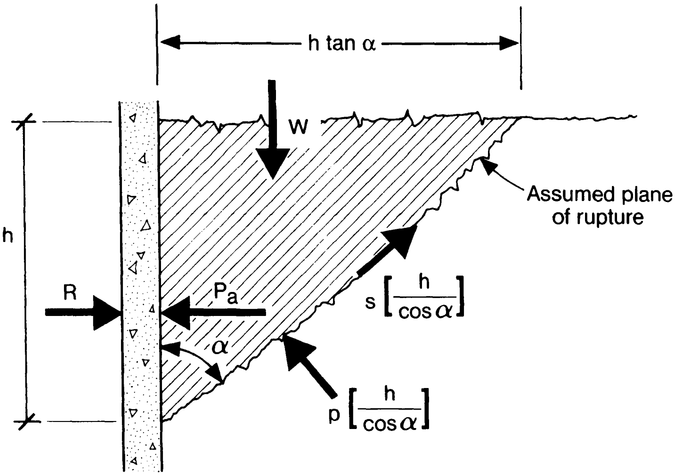


FIGURE 9-4. Free body diagram of an earth wedge on the verge of failure.

By taking the summation of forces acting vertically on the wedge:

$$W = \left[\frac{ph}{\cos\alpha}\right]\sin\alpha + \left[\frac{[p\tan\phi]\,h}{\cos\alpha}\right]\cos\alpha$$

from which: $W = ph\tan\alpha + ph\tan\phi$

By taking the summation of forces acting horizontally on the wedge:

$$R = P_a = \left[\frac{ph}{\cos\alpha}\right]\cos\alpha - \left[\frac{[p\tan\phi]\,h}{\cos\alpha}\right]\sin\alpha$$

from which: $P_a = ph - ph\tan\phi\tan\alpha$

By solving each question for (ph), P_a can be found in terms of W:

$$P_a = W\left[\frac{1 - \tan\phi\tan\alpha}{\tan\phi + \tan\alpha}\right]$$

from which:

$$P_a = \left[\frac{\gamma h^2\tan\alpha}{2}\right] \times \left[\frac{1 - \tan\phi\tan\alpha}{\tan\phi + \tan\alpha}\right]$$

then:

$$P_a = \frac{1}{2}\gamma\,\mathrm{h}^2\left[\frac{1 - \tan\phi\tan\alpha}{\dfrac{\tan\phi}{\tan\alpha} + 1}\right] \quad (9\text{-}1)$$

9-5. COEFFICIENT OF ACTIVE PRESSURE

P_a is a force. It is frequently referred to, however, as active pressure. This terminology was introduced in the previous article. Formula (9-1) can be rewritten in the following form:

$$P_a = \frac{1}{2} K_a \gamma h^2 \tag{9-2}$$

$$\text{in which: } K_a = \frac{1 - \tan\phi \tan\alpha}{\dfrac{\tan\phi}{\tan\alpha} + 1} \tag{9-3}$$

(K_a) is called the coefficient of active pressure. Note that this coefficient depends solely on the angle of internal friction of the soil and the angle defining the plane of rupture under consideration.

The design of a retaining wall must be based on the maximum amount of thrust to which it can be subjected. This thrust is exerted by the active pressure of the soil positioned in back of the wall. The maximum numerical value of this active pressure will occur when the coefficient of active pressure is at maximum value. For a given soil the angle of internal friction is constant. Therefore, the coefficient depends only on the angle of rupture. The maximum value of this coefficient can be found by performing the following mathematical procedure:

1. Differentiate Formula (9-3) with respect to the angle of rupture, set the derivative equal to zero, and solve for the angle.
2. Substitute the magnitude of this angle into Formula (9-3) and solve for the coefficient. This will be the maximum value, based solely on the angle of internal friction for the soil under consideration.

The numerical value of the angle of rupture as computed in item 1 is found to be:

$$\alpha = 45° - \frac{\phi}{2} \tag{9-4}$$

The mathematics of this procedure is shown in Appendix G.

In Article 4-8 the plane of rupture was determined by a theory involving Mohr's circle. The compatibility between that article and this article is demonstrated herein. In Article 4-8 the slope of the plane was identified by the angle i, which was measured from the horizontal. In the present article the slope of the plane is identified by the angle α, which is measured from the vertical. An examination of Figure 9-5 will show the compatibility between both angles. The angle of rupture can properly be expressed in either of the two ways. In this chapter, α will be used to identify the angle of rupture.

The maximum value of the coefficient of active pressure can now be found

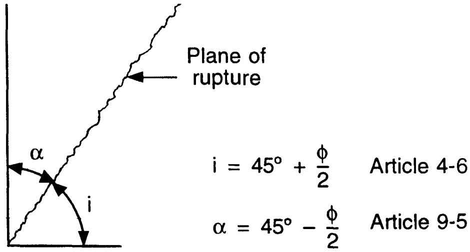


FIGURE 9-5. Compatibility of the slope of the plane of rupture as developed in Articles 4-6 and 9-5.

by substituting the above value of α into Formula (9-3) and solving. The maximum value of K_a is then found to be:

$$K_a = \tan^2\left[45° - \frac{\phi}{2}\right] \tag{9-5}$$

Formulas (9-4) and (9-5) clearly show that the angle of rupture and the coefficient of active pressure are solely dependent upon the angle of internal friction. Representative values of these properties are given in Table 9-1.

TABLE 9-1. Correlation Between Angle of Internal Friction, Angle of Rupture and the Coefficient of Active Pressure

ϕ	α	K_a
0	45	1.00
5	42.5	0.84
10	40	0.70
15	37.5	0.59
20	35	0.49
25	32.5	0.41
30	30	0.33
35	27.5	0.27
40	25	0.22

9-6. VALIDITY OF THE ACTIVE PRESSURE FORMULAS

One of the great truths of engineering is that a structure must be built to conform in all respects with the assumptions under which it was designed. Conversely, if the conditions under which the structure is to be built are known, then the design must conform to those conditions.

Formulas (9-2) and (9-5) can be used to determine accurately the numerical value of the horizontal force for which a retaining wall must be designed. They are subject only to the requirement that the work as built conforms to the assumptions under which the equations were developed, as itemized in Article 9-4.

9-7. EQUIVALENT LIQUID PRESSURE THEORY

Formula (9-2), which numerically identifies the horizontal force P_a, can be rewritten in the following form:

$$P_a = \frac{1}{2}\,[K_a\,\gamma\,h]\,h \tag{9-6}$$

in which the term $[K_a \gamma h]$ represents the intensity of pressure at the base of the wall.

The total force P_a may then be represented geometrically by the area of a triangle whose base is the intensity of pressure and whose height is the height of the wall. This is the same procedure by which pressures and forces are computed for liquids. It is for this reason that this procedure, when applied to soils, is referred to as the equivalent liquid pressure theory. The examples enumerated in Table 9-2 illustrate the use of this theory for the more frequently encountered conditions of loading (Figs. 9-6 through 9-9).

TABLE 9-2. Common Sources of Active Pressure

Type	Earth Surface	Surcharge	Water Table	Figure No.
1	Level	No	No	9–6
2	Level	Yes	No	9–7
3	Level	No	Yes	9–8
4	Sloped	No	No	9–9

9-8. NUMERICAL ACCURACY OF K_a AND P_a

The validity of all computations involving the equivalent liquid pressure theory depends on the accuracy of the numerical values of the following two things:

1. *The coefficient of active pressure K_a:* This coefficient, as shown in Article 9-6, is solely dependent on the magnitude of the angle of internal friction of the soil which the wall is to support. Values for K_a based on this dependency are given in Table 9-1.

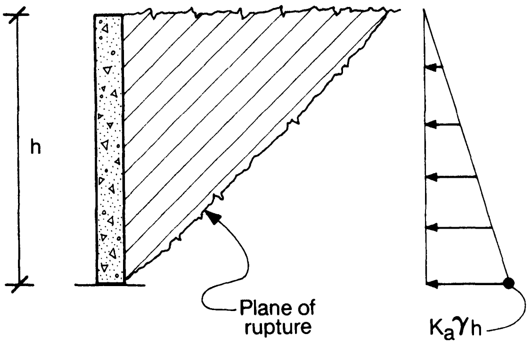


FIGURE 9-6. Pressure diagram produced solely by earth.

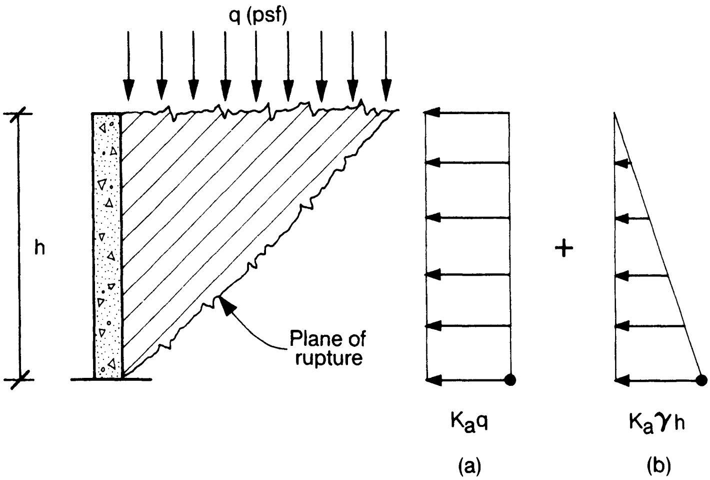


FIGURE 9-7. Pressure diagrams produced by (a) surcharge, and (b) earth.

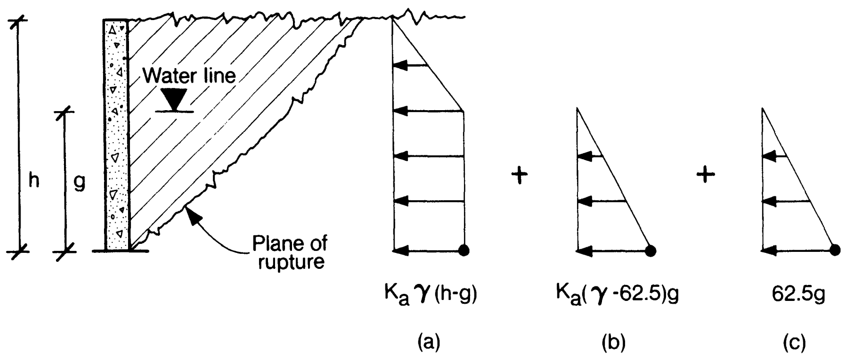


FIGURE 9-8. Pressure diagrams produced by (a) earth, (b) submerged earth and (c) water.

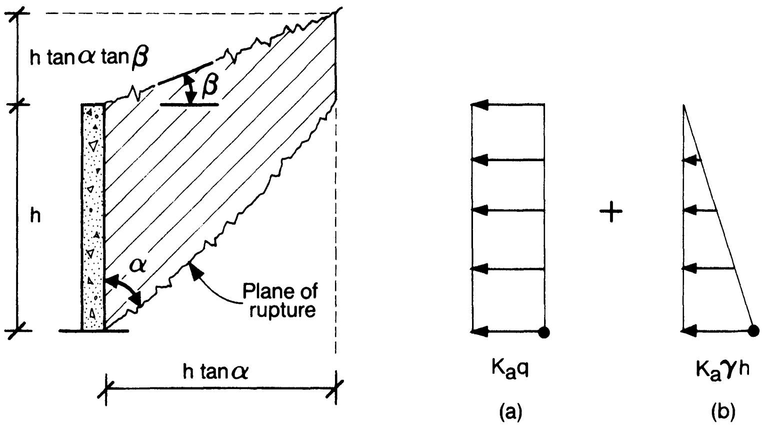


FIGURE 9-9. Pressure diagrams produced by sloping earth, where (a) is due to the earth above the wall (acting as a surcharge), and (b) is due to the earth in back of the wall. The surcharge $q = 0.5\gamma\ [h \tan\alpha \tan\beta]$.

2. *The horizontal force P_a:* This force is dependent on the value of the coefficient of active pressure and on the unit weight of the soil. The coefficient can be approximated as noted in item 1 above. The unit weight of the soil can be approximated from the values given in Tables 2-1, 2-2, and 12-3.

The above noted approximations are just that—approximations. The architect or engineer in charge of the project must determine whether these approximations are sufficiently accurate for the work at hand. Under normal circumstances the values thus determined for K_a and P_a could be used for preliminary design. Final design, however, should not normally rely on these approximations but should rely on the results of a laboratory analysis in which accurate values for the required properties can readily be determined.

It must be recognized that in almost all areas of soil mechanics there are unknowns, intangibles, and inconsistencies that cannot be uncovered or understood without benefit of some type of engineering investigation. A formal investigation, including field and laboratory work, has been outlined in Article 3-4. The architect or engineer, in consultation with the owner (who pays the bills), must determine the extent of any required investigation. Such an investigation will establish realistic pressures for which the retaining system can be designed and will also uncover any problems that should be considered during the design or construction stage of the project.

It has been the experience of the author that such a program of subsurface exploration can be of significant value in the overall operation of projects of any size or complexity. It is strongly recommended that some kind of program be initiated in all such projects and in any other project where the architect and engineer wish to base their design on factual and conclusive information.

9-9. CHARTS FOR ESTIMATING BACKFILL PRESSURE

Reasonable estimates of the earth pressure acting on a wall can be made by use of the charts given in Figure 9-10. These charts are based partly on theory and partly on studies of the performance of satisfactory and unsatisfactory walls. It must be emphasized, however, that these charts give estimates only. An accurate analysis and design can only be made based upon the results of a subsurface soils exploration.

It is recommended that the use of these charts be limited to walls whose height does not exceed 20 feet.

In order to use these charts, the material that will be used for backfill must first be classified in one of the five listed categories:

1. Coarse-grained soil without admixture of fine particles, very permeable, as clean gravel or sand
2. Coarse-grained soil of low permeability due to admixture of particles of silt size

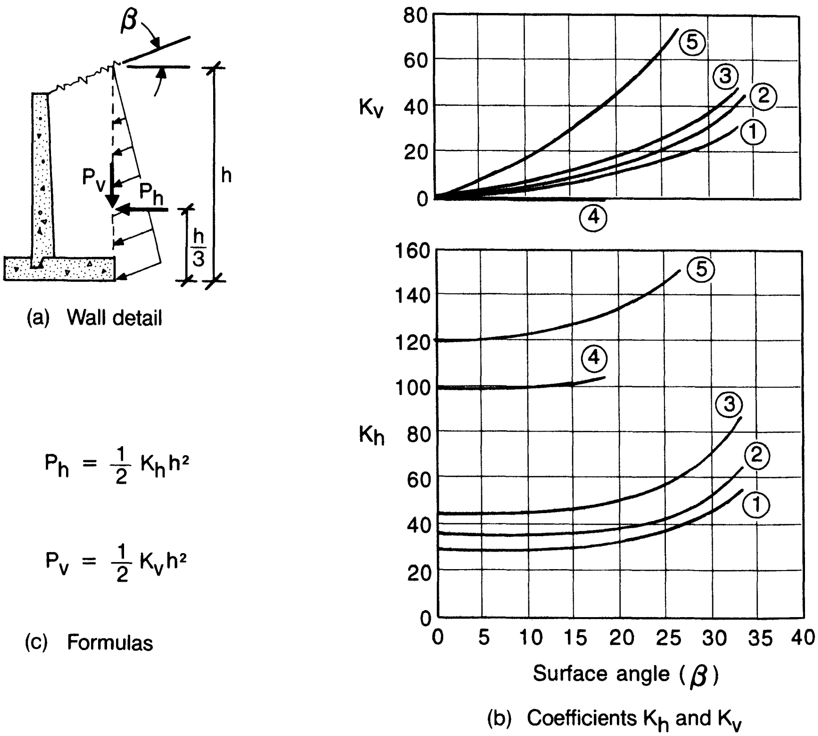


FIGURE 9-10. Charts for estimating backfill pressure. [Ref. 20]

3. Fine silty sand, granular materials with conspicuous clay content, and residual soil with stones
4. Very soft or soft clay, organic silt, or silty clay
5. Medium or stiff clay, deposited in chunks and protected in such a way that a negligible amount of water enters the spaces between the chunks during floods or heavy rains; otherwise, the clay should not be used for backfill—with increasing stiffness of the clay, danger to the wall due to infiltration of water increases rapidly

These charts give values for the coefficient (K_h), instead of K_a. These coefficients are related as follows:

$$K_h = K_a \gamma$$

Where:
K_h = the coefficient of horizontal pressure
γ = the unit weight of the in-place earth

These charts also give values for a coefficient symbolized by (K_v). This coefficient is used in cantilever retaining wall analysis. It takes into consideration a beneficial resistance to shear that can be developed within the earth mass on a vertical plane passing through the back edge of the footing. As the retaining wall rotates in a failure mode, it can be seen that the earth that bears on the footing must lift. In order for this earth to lift it must shear away from the earth situated beyond the footing. As a result this shear acts as a restraining force to assist in stabilizing the retaining wall against rotation and subsequent failure.

This shear, which is developed between the two masses of earth, is a function of intergranular friction and cohesion. It is given analytically in terms of the coefficient K_v.

The decision to incorporate the restraining effect of this shear into computations based on these charts is left to the individual designer. Since most engineers tend to be conservative when dealing with soils, the author recommends that this shear be ignored in the computations.

Numerical values for the coefficients K_h and K_v may be taken directly from the charts given in the referenced figure.

9-10. SAMPLE PROBLEMS

Note: When investigating or designing walls subjected to lateral earth pressure, it should be remembered that all calculations are based on one linear foot of wall.

Example 9-1

Required: To determine the pressure gradient and total force acting against a 14-foot-high wall, subject to the following:

From laboratory analysis: $\gamma = 110$ pcf and $\phi = 32°$

From Table 9-1: $K_a = 0.31$

Refer to Figure 9-6 for the method and to Figure 9-11 for the solution.

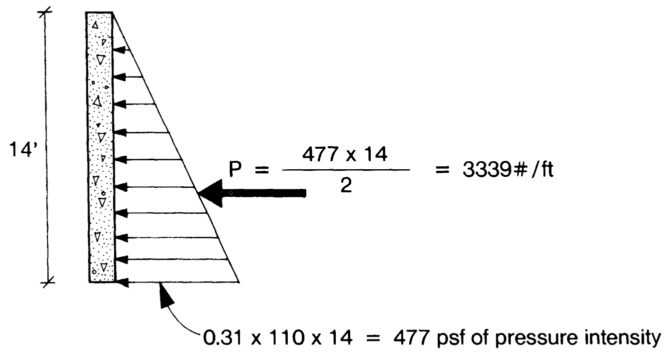


FIGURE 9-11. Example 9-1—Lateral pressure from backfill only.

Example 9-2

Required: Repeat Example 9-1, but add a surcharge of 400 psf.

Refer to Figure 9-7 for the method and to Figure 9-12 for the solution.

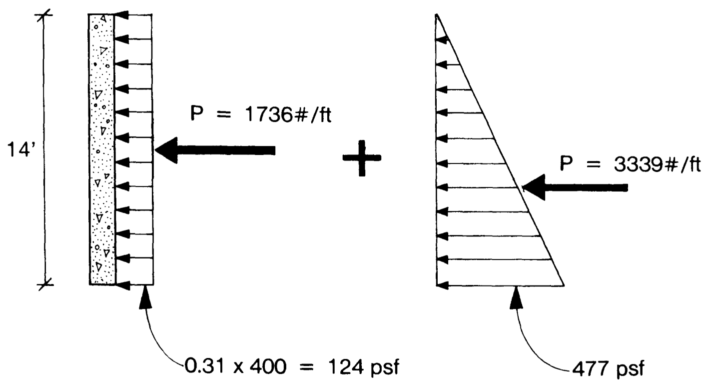


FIGURE 9-12. Example 9-2—Lateral pressure with surcharge.

Example 9-3.

Required: Repeat Example 9-1, but add a water table 6 feet above the base.

Refer to Figure 9-8 for the method and to Figure 9-13 for the solution.

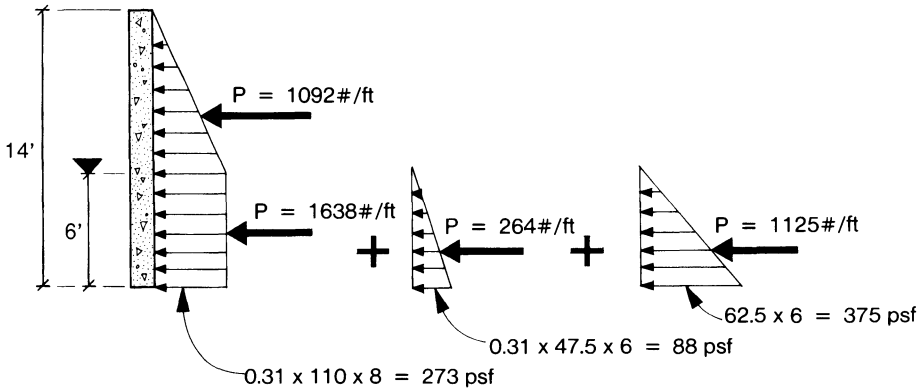


FIGURE 9-13. Example 9-3—Lateral pressure with water table.

Example 9-4

Required: Repeat Example 9-1, but slope the surface of the backfill 30°.

From Table 9-1: $\alpha = 29°$

Refer to Figure 9-9 for the method and to Figure 9-14 for the solution. In solving this problem the sloping earth is treated like a surcharge whose intensity equals the average weight of the earth. Therefore:

$$q = 0.5 \times 110 \times 14 \times \tan 29° \tan 30° = 246 \text{ psf}$$

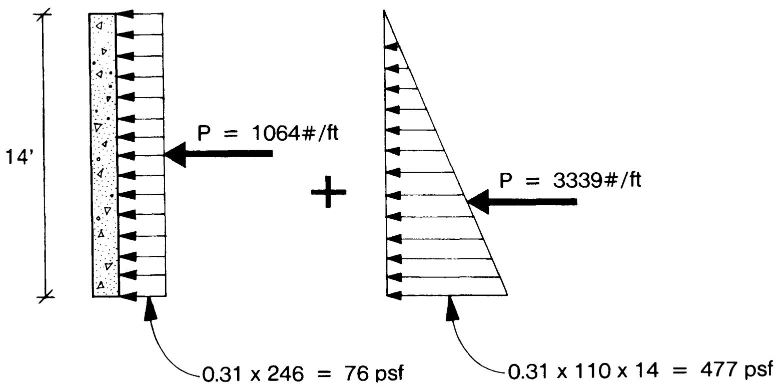


FIGURE 9-14. Example 9-4—Lateral pressure with sloping backfill.

Example 9-5

Required: To determine the total force acting against a 14-foot-high wall by using the charts for estimating backfill pressure. Assume the backfill to be type 2, with a level surface.

Refer to Figure 9-10 for the method and to Figure 9-15 for the solution.

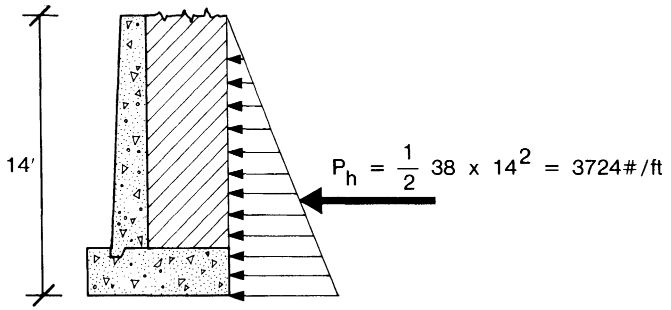


FIGURE 9-15. Example 9-5—Lateral pressure using charts with level backfill.

Example 9-6

Required: Repeat Example 9-5, but slope the surface of the backfill 30°.

Refer to Figure 9-10 for the method and to Figure 9-16 for the solution.

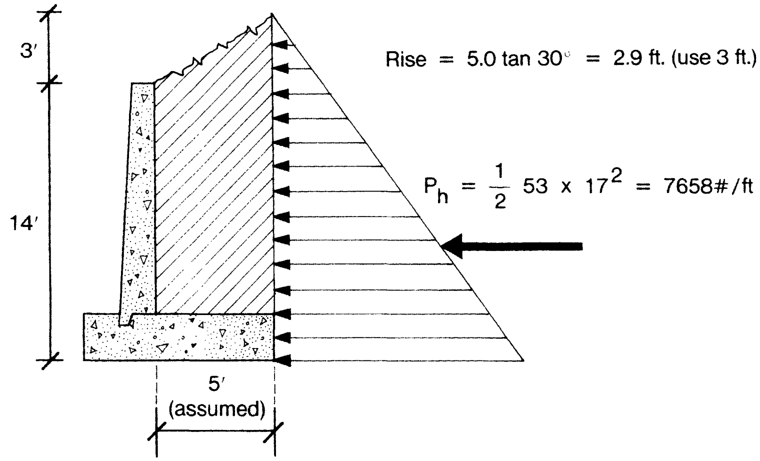


FIGURE 9-16. Example 9-6—Lateral pressure using charts with sloping backfill.

10

Walls—Construction Details

10-1. GENERAL

There are times in the course of the architectural development of a project, or during the construction of the project itself, when it is necessary to alter the contours of the existing grade. These changes may be temporary, or they may be permanent. Examples of such changes are as follows:

1. In the construction of highways, particularly at ramps, bridges, and crossovers
2. Where bulkheads are required for the control of water and land erosion in coastal areas
3. As part of the architectural treatment of the overall development of the site
4. Where temporary working space is required in which to construct the underground areas of a building or other structure
5. At the exterior walls of basements of buildings and at interior areas such as elevator pits, pipe tunnels and other mechanical spaces

Changes in grade can be produced by reshaping the surface of the earth to form an embankment, or by constructing a physical barrier such as a basement wall or a free standing retaining wall. The ways in which these changes are routinely made in everyday construction are illustrated in the following paragraphs.

10-2. EARTH EMBANKMENT

Earth embankments are a relatively easy, quick and inexpensive way of providing a change in grade. The work is done by grading the earth to a stable slope at,

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or preferably somewhat less than, the angle of repose of the soil. Steeper banks can be constructed using a process called lime stabilization in which lime (calcium hydroxide) is mixed with the soil before placement and compaction. Another process called lime injection can be used to stabilize existing embankments. The embankment must be protected against surface erosion, landslide or rutting as caused by wind or rain. In embankments having a low rise to length ratio, it may be possible to provide adequate protection by planting a good stand of grass or other suitable ground cover. Such protection would normally be adequate for the low level of slope found in residential work or for playgrounds. For steeper slopes special care is required to avoid the incidence of soil failure as illustrated in Figure 10-1.

For embankments with steep slopes, or for those used in highway construction, a more positive and maintainance free kind of protection is required. Such protection is usually provided by covering the surface with a reinforced concrete slab. A typical example of such a protection is shown in Figure 10-2. A finish coat of brick or stone laid in cement can be built on top of the concrete slab if desired for aesthetic considerations. Because these slopes are relatively steep there is a tendency on the part of the concrete slab to slide down the slope. Such slabs may be anchored into the soil with short lengths of piles or with poured-in-place turndowns of the slab. The design of these slabs must also address the problem of thermal expansion and contraction, and the adverse action of heaving of the subgrade due to frost.

FIGURE 10-1. Typical sliding failure of an earth embankment.

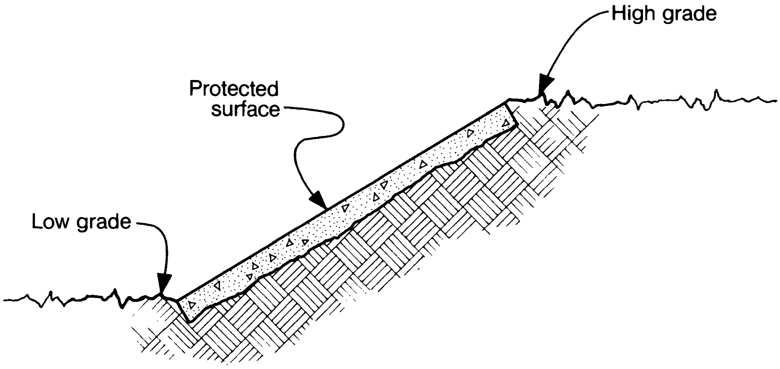


FIGURE 10-2. Earth embankment stabilized with a concrete cover.

10-3. SHEET PILING RETAINING WALLS

A typical detail of a steel sheet piling wall is illustrated in Figure 10-3. The elements of this wall are sheet piling, wales, tiebacks, and grout pockets. The sheet piling is manufactured of high strength steel and is available in various cross sections and thicknesses. The piling must be designed not only to resist earth pressure but also the dynamic forces induced by the driving operation. Wales are horizontal members placed hard against the outer face of the sheet piling. Their purpose is to support the piling and transfer the lateral earth pressure by beam action to the tiebacks. Wales are usually of heavy timber but could also be of structural steel. Tiebacks are prestressed cables extending to some point behind the wall where they are anchored into the underlying bedrock or undisturbed earth. Their purpose is to provide lateral support to the wales. For details

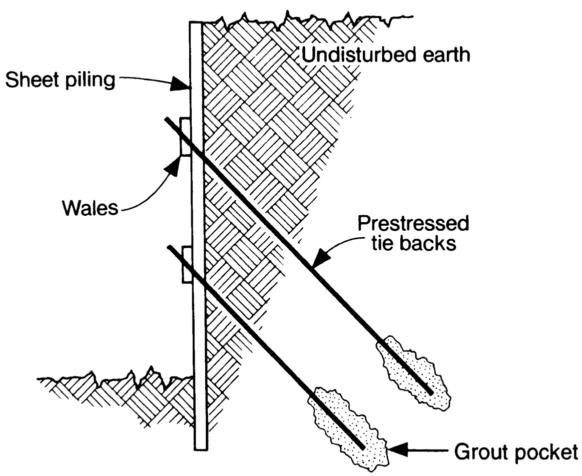


FIGURE 10-3. Typical detail of a steel sheet piling retaining wall.

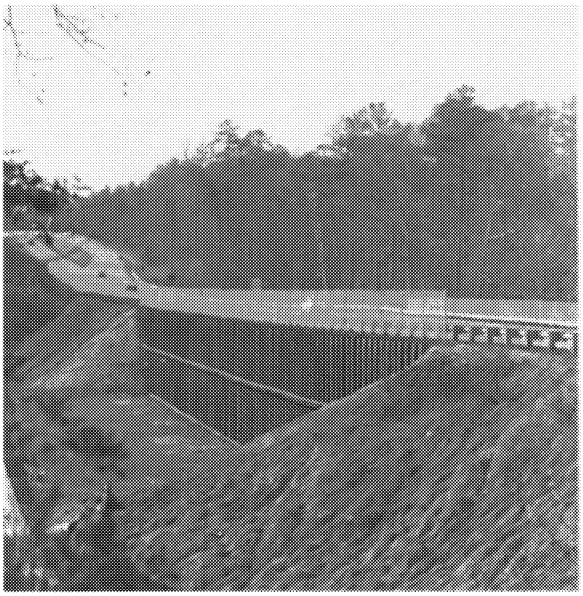


FIGURE 10-4. Bethlehem PZ27 sheet piling stabilizes this busy roadway in a marshy location near Williamsburg, Va. [Ref. 3]

relative to the design and installation of these cables and grout pockets refer to Articles 10-11 and 10-12 and Examples 11-9 and 11-10.

There are two ways to construct this type of wall, depending on whether the wall is to lower grade or raise grade. In the former case the wall is constructed by driving the sheet piling into unexcavated ground with heavy, machine driven hammers. After the piling has been driven to the specified depth, the earth on the side to be exposed is partially excavated, and the upper level of wales and tiebacks are installed. This procedure is continued in stages until construction is complete. In the latter case the piling is erected in open ground and temporarily held in place while the permanent tieback system and earth backfill is placed.

Sheet piling can effectively be used aesthetically, as illustrated in Figure 10-4. Note that in this figure the connection between the cables and the wales is clearly visible.

10-4. SOLDIER BEAM RETAINING WALLS

A typical detail of a soldier beam wall is illustrated in Figure 10-5. Soldier beam walls are used in situations where major changes in grade are required. Such walls may be temporary, or they may be permanent. A prime example of the use of this type of wall is where deep excavation is required for the installation

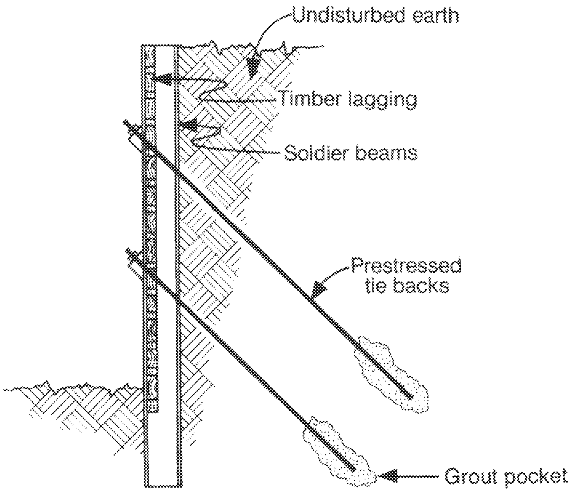


FIGURE 10-5. Typical detail of a soldier beam retaining wall stabilized with tiebacks.

of permanent construction such as a building, and where the integrity of the upper grade must be preserved during all phases of the work.

Soldier beams are steel HP sections which are similar in cross section to W sections, except that their web thickness has been increased to that of the flanges in order to provide equal resistance to buckling while being driven. HP sections were first introduced in Article 8-5. Soldier beams are placed approximately eight foot on center along the length of the wall. The term soldier beams was derived from the fact that, after installation, these beams resemble a row of soldiers all standing in line at attention. Soldier beams are driven into unexcavated ground with heavy, machine driven hammers to a predetermined depth, which is usually bedrock. The earth in front of the soldiers is then partially excavated, and the upper level of timber lagging and tiebacks are installed. This procedure is continued in stages until construction has been completed. Lagging consists of creosoted timber planks which are usually cut to fit between the webs of the solder beams and placed hard against the outside face of the inside flange. The purpose of the lagging is to restrain the earth and transfer the lateral earth pressure to the soldier beams. Lagging produces a reasonably tight fit, but there will be numerous gaps and cracks through which water and soil can escape. In order to impede this movement, lagging is frequently interlaced with thick layers of straw. Tiebacks provide lateral support to the soldier beams. Note that each soldier beam must be supported by a tieback.

Soldier beams can be stabilized against lateral forces by methods other than tiebacks. In instances where the excavation is linear and relatively narrow, as in the construction of tunnels, subways and sewers, the soldier beams can be braced across the excavation with horizontal compression struts. In large excavations, where cross bracing becomes impractical, then inclined shores called rakers may

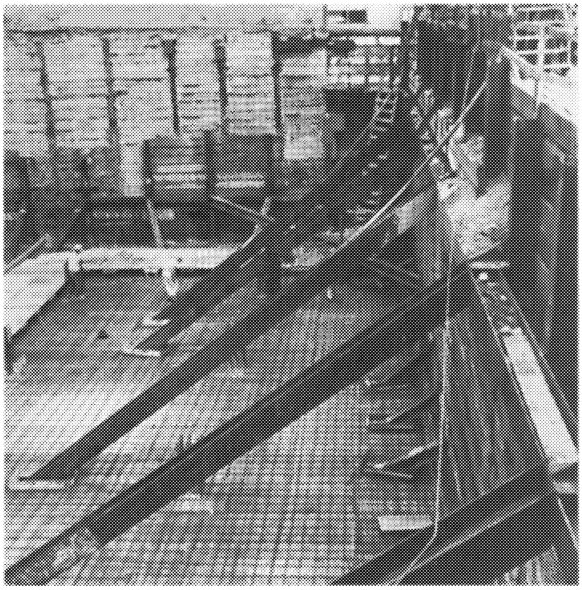


FIGURE 10-6. A soldier beam wall stabilized with wales and inclined shores called rakers. [Ref. 3]

be used. Such an installation is illustrated in Figure 10-6. This system of providing for a deep excavation is frequently used, particularly because of the relative economy compared to that of tiebacks. It is also a very positive method and is readily adjustable. Occasionally the soil in back of the wall is not suitable for a tieback installation in which case the raker method must be used. The main disadvantage to this system, of course, is the potential for interference with subsequent construction and the work of other building trades.

Note in Figure 10-6 that the timber lagging has been placed outside of the soldier beams rather than within the webs, as in Figure 10-5.

Whereas the tieback system is frequently used as a permanent bracing system both the cross bracing and raker systems are fundamentally temporary installations.

10-5. SITE PREPARATION FOR BUILDING CONSTRUCTION

The construction of a major building project will begin with preparation of the site. Most buildings have extensive usable basement areas below grade, therefore, extensive excavation will be required. The primary bulk excavation is usually brought down to subgrade, defined as the underside of the basement slab on ground. This may be done in small sections with the construction of foundations

FIGURE 10-7. A typical site during construction of a major project.

commencing in that area, or the entire site may be excavated prior to any other construction. This work is usually done with heavy duty earth moving equipment. Secondary excavation for elevator pits, spread footings, piers and the like will follow as needed. The scheduling of all work is solely the prerogative of the general contractor. A site in progress is illustrated in Figure 10-7. It can be seen that the bulk excavation has been completed. Secondary excavation for pits and piers is in progress. The access ramp, so necessary to the construction process, was formed out of earth spoils from the excavation.

10-6. BASEMENT WALL—GENERAL CONDITIONS

The purpose of a basement wall is to provide an enclosure for a usable building space constructed below grade. The basement wall serves the dual function of restraining earth and ground water from entering the building, and of transferring building loads to the supporting foundations.

Basement walls are almost always cast-in-place, although there are instances of walls having been precast and set in place. A typical detail of a cast-in-place basement wall is shown in Figure 10-8. Special conditions which may occur at the top or bottom of the wall are shown in subsequent details.

It should be noted that basement walls and slab on ground should be protected

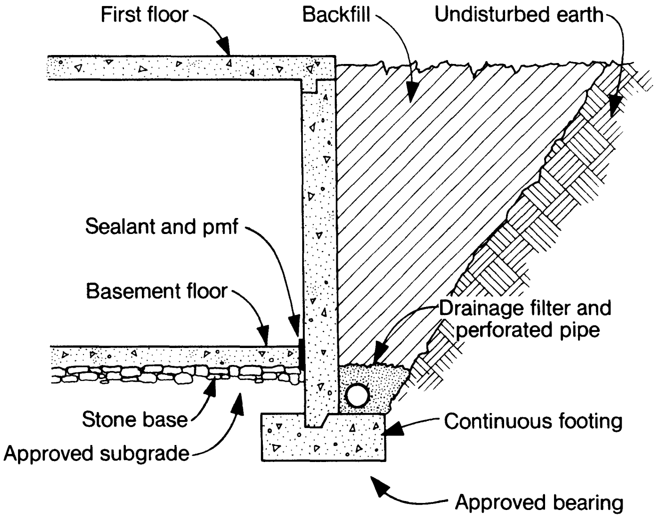


FIGURE 10-8. Typical detail of a cast-in-place basement wall.

from leakage of water and/or influx of moisture. This protection has not been shown in these details as it is considered to be beyond the scope of this text.

In order to construct this wall and the adjacent basement areas, the contractor will require a large, cleared space in which to work. This is the purpose of site excavation. This excavation will be brought to subgrade by the use of heavy machinery. Care must be taken, however, not to extend the excavation below the top of the footing.

After excavation has been completed, wood forms called screed rails, as described in Article 7-2. are then set to the exact elevation of the top of the footing. The excavation for the footing is then dug and trimmed by hand, reinforcing is installed, and the footing is poured. After the concrete has hardened, the construction of the formwork for the walls can be started. The placement of formwork for a foundation wall is shown in Figure 10-9, and the subsequent pouring of the wall is shown in Figure 10-10. Note that in this particular instance the concrete truck has access to the wall and concreting can be made directly from the truck through a shute.

It should be noted that after the wall has been poured the exterior backfill can not be placed until the permanent elements supporting the lateral pressure of this backfill have been installed. For those rare instances when it is desirable or necessary to place the backfill before installing these restraining elements, the wall must then be temporarily supported. This is accomplished by using either of the following methods:

1. Tie the wall back with prestressed cables extending back into the undisturbed earth on the exterior side of the wall, as shown in Figure 10-5.

FIGURE 10-9. Formwork for a foundation wall in progress.

FIGURE 10-10. Concreting a foundation wall directly from the truck using a shute.

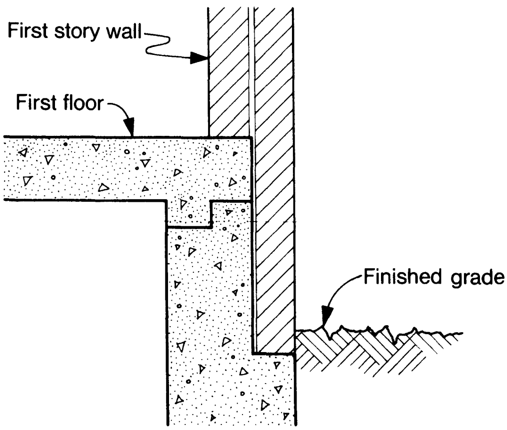


FIGURE 10-11. Basement wall with face brick below grade.

2. Construct timber or steel shoring from the wall down to the subgrade on the basement side of the wall, as shown in Figure 10-6.

Both methods of temporary support are expensive and time consuming and should only be used after alternatives have been carefully considered.

10-7. BASEMENT WALL—TREATMENT AT GRADE

Face Brick Extending Below Grade

The detail shown in Figure 10-11 depicts a brick faced building in which the face brick extends below grade. The face brick is supported by a ledge built into the concrete wall. This ledge, commonly called a brick shelf, should be placed in coursing several courses below grade. Coursing is a term related to brickwork in which one course is one height of brick plus a mortar joint. Three bricks and three mortar joints generally course to 8 inches. When brickwork is to be built on a concrete shelf, the shelf should line up with the top of a brick. The shelf is normally specified to be four and one-half to five inches in width in order to allow sufficient lateral space for mortar and flashing. The basement wall should be a minimum of twelve inches thick at its base because of the reduced thickness behind the brick shelf.

This detail is also applicable when other facing materials, such as granite, marble or precast units, are used. The width and elevation of the supporting shelf must be adjusted accordingly.

Concrete Wall extending above Grade

The detail shown in Figure 10-12 is applicable when it is desired to expose the exterior face of the concrete wall. The first floor wall is located inward so that

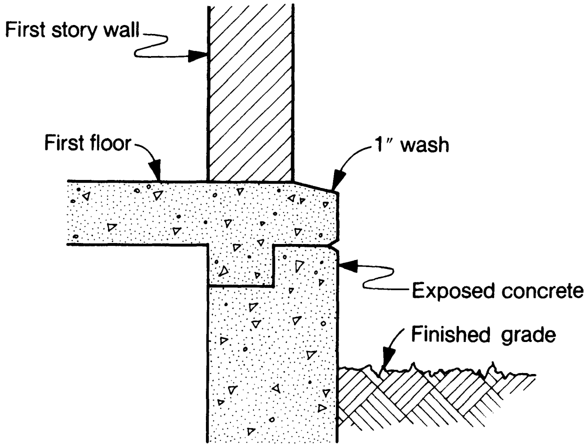


FIGURE 10-12. Basement wall with concrete extending above grade.

the exposed edge of the concrete slab can be beveled to form a wash. The purpose of this wash is to provide for the control of water where the first floor wall meets the concrete.

Special attention should be directed by the architect to the kind of finish to be specified for the exposed face of the concrete. He should also consider the desirability of accentuating the horizontal line joining the concrete wall and the slab which otherwise may appear as an unsightly crack.

Recessed Entrance

The detail shown in Figure 10-13 applies where the first floor wall has been moved inward from the face of the basement wall in order to create a recessed

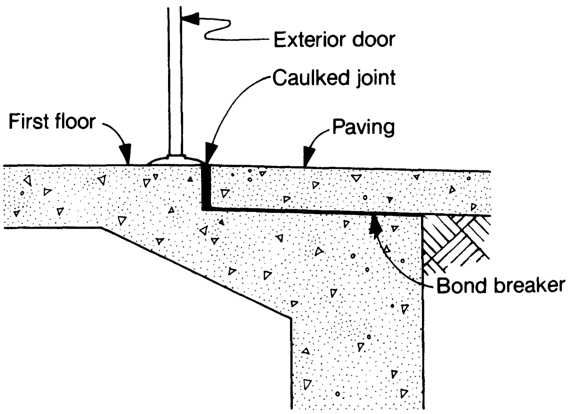


FIGURE 10-13. Basement wall with a recessed entrance and exterior paving.

entrance or because of other architectural considerations. Assuming that the threshold of the door is not directly exposed to rain water, the paving can be at the same elevation as the first floor as shown in the detail. This is a desirable detail for an entrance accessible to the handicapped, in which case the entrance should be protected from direct rain water with some kind of canopy. When the threshold is directly exposed to rain water, the paving should then be depressed the depth of one step. In either case the paving should be pitched no less than ¼″ per foot away from the building in order to provide for the positive runoff of water. Note that the bottom of the interior floor slab must be thickened or haunched to replace the strength lost due to the recess.

As detailed, the basement wall provides a positive support for the exterior paving. If desired, this paving could be made a structural slab with additional supports beyond the building. This slab could also be depressed to provide for a brick or stone walking surface.

For the detail as shown, it is difficult to avoid having a crack develop in the paving at the outer face of the foundation wall. Additional reinforcing should be installed at the top and bottom of the slab. Architects frequently specify that the surface of the slab be scored where the crack is expected to occur.

First Floor Slab Extension

Figure 10-14 illustrates the result of extending the first floor slab beyond the building to create a loading dock or some other form of overhang. The slab extension should be depressed and should be pitched away from the building no less than ¼″ per foot to provide for the positive control of water. The vertical distance between the top of the overhanging slab and the underside of the first floor slab must be carefully coordinated so that the tensile reinforcing in the overhang can extend back into the floor slab a distance sufficient to develop the required anchorage. It may be necessary to provide a thickening of the interior floor slab in order to provide cover for this extension.

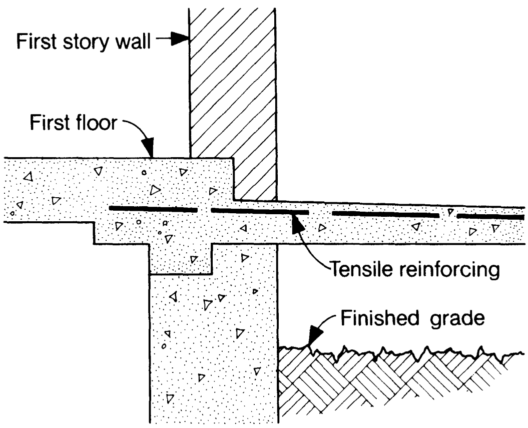


FIGURE 10-14. Basement wall with an extended first floor slab.

10-8. BASEMENT WALL—TREATMENT AT BASE

Basement walls not only restrain earth, they also carry gravity loads. The thrust of the basement wall can be transferred laterally to the earth by a continuous footing or to the slab on ground by direct bearing. Gravity loads can be transferred to the earth by a continuous footing or by a series of intermittent footings. In any case, whether it be passive pressure at the side of a footing, frictional resistance at the base of the slab on ground, or direct bearing beneath a footing, it is interesting that the ultimate response to the loads imposed on a building must be provided by the earth.

Basement Wall Bearing on a Continuous Footing

Basement walls are preferably built directly on a continuous footing. This method of construction is very practical and should be used whenever the soil is capable of providing adequate support for the footing within a depth of three to four feet below the slab on ground.

The detail shown in Figure 10-15 illustrates the construction where the basement wall bears on and receives vertical and horizontal support from a continuous wall footing. Since there is no transfer of lateral earth pressure from the wall to the slab, the slab may be separated from the wall with a premolded filler (pmf) and a sealant. The slab may even be poured at a later date.

Gravity load transfer is made through direct bearing to the footing. Lateral load transfer is made through a shear key. The shear key is formed by building a wood block into the footing prior to concreting. After concreting, the wood block is removed. Note that the inward face of the key is vertical. This is because that face transfers the load through direct bearing. The outer face, however, is sloped. This is to facilitate removal of the wood block. For details and allowable load transfers on shear keys, refer to Article 11-7 and Appendix B.

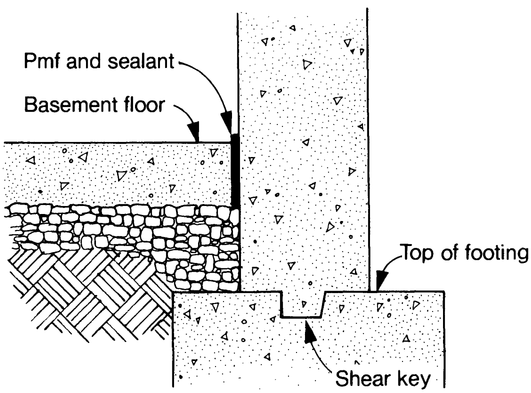


FIGURE 10-15. Earth pressure transfer from basement wall to footing.

Slabs on ground which do not receive earth pressure need only have sufficient thickness and reinforcing to transfer the superimposed floor loads directly to the ground without sustaining cracks, misalignment or other signs of distress. Slabs in this category should be no less than four inches thick even when lightly loaded. For recommended thickness and reinforcing of slabs on ground for all ordinary loading conditions, refer to Appendix D.

Basement Wall Designed as a Grade Beam

The method of using a continuous wall footing becomes less and less practical as the footing is lowered to reach adequate soil bearing pressure. There will come a point at which it is no longer feasible or cost effective to use a continuous footing. In that case the wall must be designed to span as a beam between isolated foundation elements which are extended down to adequate bearing. Such a wall is commonly called a grade beam. When architectural design has positioned main building columns in the exterior wall, the grade beam can then be designed to span between these columns. When building columns have not been placed within the wall, other supports must be provided. These supports may be piers bearing on spread footings, caissons or piles as indicated by soil analysis. Such supports are usually spaced about 20 to 30 feet on center along the wall, subject to the type of structural system being used.

The detail shown in Figure 10-16 is representative of grade beam construction. Even though grade beams do not require footings, the contractor may elect to pour a thin pad of concrete under the grade beam to provide a solid platform upon which to set forms and to otherwise facilitate construction.

Earth pressure, in this instance, is transferred from the wall to the basement slab. Note that such a transfer can only be made through physical contact between the wall and the slab. The premolded filler of the previous example cannot be used here. It is very important to understand that in this method of transfer the

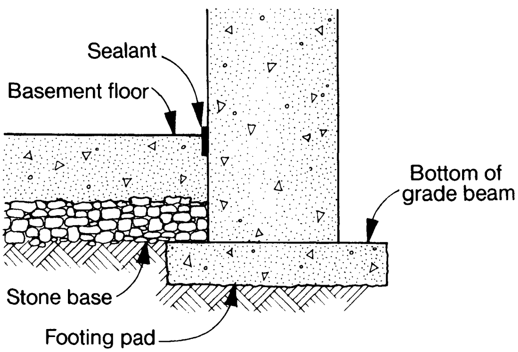


FIGURE 10-16. Earth pressure transfer from basement wall to slab on ground using grade beam construction.

wall cannot be backfilled until the slab has been poured and has attained its required strength.

It is recommended that whenever the slab on ground is used to transfer earth pressure, it should have a thickness no less than five inches even for lightly loaded walls. The contractor usually experiences difficulty in pouring the slab on ground due to a certain amount of instability of the working area. There is particular difficulty in maintaining the proper elevation of the screeds, reinforcing steel and other built-in items. Because this slab has the dual function of transferring earth pressure as well as serving as a finished basement floor, the author believes that a little extra thickness should not be considered unreasonable.

10-9. GRAVITY-RETAINING WALLS

Gravity-retaining walls are earth retaining elements consisting of a solid mass. Concrete is the most frequently used material for this kind of wall. Stone or some other kind of masonry could be used provided that the wall is solid. This requires that the individual pieces of stone or masonry be cemented together and that all voids between the pieces be filled with cement grout. Concrete walls are usually lightly reinforced with vertical and horizontal bars placed on the exposed face of the wall. This reinforcing is rather nominal and is for the purpose of controlling cracks, not for adding tensile strength. Concrete walls may be faced with stone or other facing material, as required by the architectural treatment of the area. A typical gravity wall is illustrated in Figure 10-17.

Gravity retaining walls are relatively easy to build because they are essentially just a solid mass of concrete. These walls depend solely on their own dead weight for stability. For this reason, the width of the wall at its base must be carefully

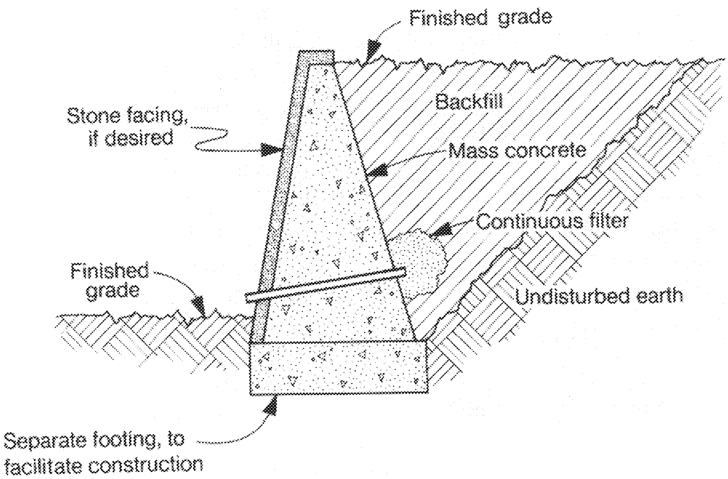


FIGURE 10-17. Typical detail of a gravity retaining wall.

computed to provide adequate resistance against failure by overturning or by slide. For a preliminary analysis a good rule of thumb is to assume the width of the base equal to one-half of the overall height of the wall. This rule of thumb is valid only when the wall retains level earth without surcharge. When lateral pressure is increased because of sloping earth or surcharge, the width of the base must be increased accordingly.

Walls whose footings are close to the water table must be carefully designed from the standpoint of slide because water leakage beneath the base may act as a lubricant to reduce or completely destroy the resistance that would normally be developed by friction and cohesion. In this instance it is quite likely that the wall must be extended farther down into the undisturbed earth in order to ensure development of sufficient passive pressure to overcome the effects of slide.

For all practical purposes this type of retaining wall should be limited to about 12 feet in height. The width of the base will be about one-half the height. The thickness at the top is limited only by the clearance necessary for the installation of concrete.

10-10. CANTILEVER RETAINING WALL

Cantilever retaining walls are a time honored way of providing for abrupt changes in grade. They are found along highways, around buildings and in a variety of landscaping and site development projects. Properly engineered, these walls give many years of satisfactory service and can be utilized through a considerable range of heights. Walls of 15 to 20 feet are not uncommon, and this height can be dramatically increased when the walls are prestressed. The author, for example, has designed walls having a height of approximately fifty feet by the use of post-tensioning.

A typical cantilever retaining wall with identification of the terminology normally associated with it is illustrated in Figure 10-18.

Cantilever retaining walls are self supporting and restrain the earth by reason of their geometry and mass. Because these walls are self-supporting, backfilling operations can proceed as soon as the concrete has attained its required strength.

The exposed face of the wall can be left as finished concrete, or it can be faced with brick, stone, or other material at the option of the architect. Either face of the wall can be battered as directed by aesthetic or engineering considerations.

The width of footing required for this type of wall is large compared to that required for other types of wall. This width depends on the following factors:

1. The in-place character and density of the backfill
2. The surface of the restrained earth, which may be level or sloped
3. The absence or existence of surcharge
4. The existance of a water table within the height of the wall
5. The positioning of the wall with respect to the footing

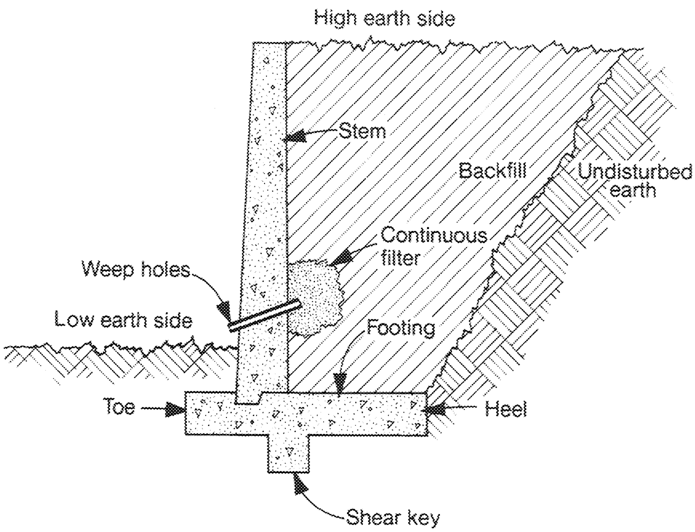


FIGURE 10-18. Typical detail of a cantilever retaining wall.

It must be noted that this is a sophisticated wall, requiring expertise in its design and construction.

10-11. WALLS REQUIRING SPECIAL RESTRAINTS

Some cantilever retaining walls simply cannot develop sufficient lateral resistance through the usual combination of base friction and passive pressure. Very high walls and walls that must support large lateral pressure due to adverse backfill, surcharge or high water table are examples of walls requiring special systems of restraint. The amount of available resistance can be dramatically increased by the use of:

1. Battered piles
2. Tiedowns
3. Tiebacks

Battered Piles

Battered piles are an effective way of providing additional lateral resistance to a wall subject to the amount of resistance required. Although this system is very practical and cost effective, its capacity is limited.

Battered piles are frequently used in granular type soils because such soils are very receptive to pile installation. The footing, therefore, may be supported with two parallel rows of timber piles or with hollow shell concrete filled piles.

Because piles act with much more assurance when in compression, it is important that the piles on the low earth side of the wall are the ones that are battered. The horizontal component of the battered piles provides the required lateral resistance. When battered piles are used, it is recommended that all the required lateral resistance should be provided by the piles without consideration of base friction or passive pressure.

The minimum spacing of piles is usually about 4 feet on center along the length of the wall. When determining the resistance to lateral load, it must be remembered to divide the horizontal component of the piles by the spacing of the piles, in order to obtain a resistance in pounds per foot of wall.

Prior to using this method of analysis, the designer should discuss this problem with several pile driving contractors who show interest in bidding the work. Not all contractors can install battered piles, and some can provide more batter than others. The design of these piles must be consistent with the capability of the contractor to install them. A battered pile installation is illustrated in Figure 8-3, and their usage is illustrated in Figure 10-19.

In soils not conducive to the driving of piles, such as stiff clay, an alternate solution would be the use of piers, which are drilled. The same cautions and recommendations presented for piles apply equally for piers.

Tiedowns

The second system whereby lateral load resistance can be increased is to anchor the footing into the soil with prestressed tiedowns. This method is highly effective when the tiedowns can be anchored into bedrock or into soil that is extremely

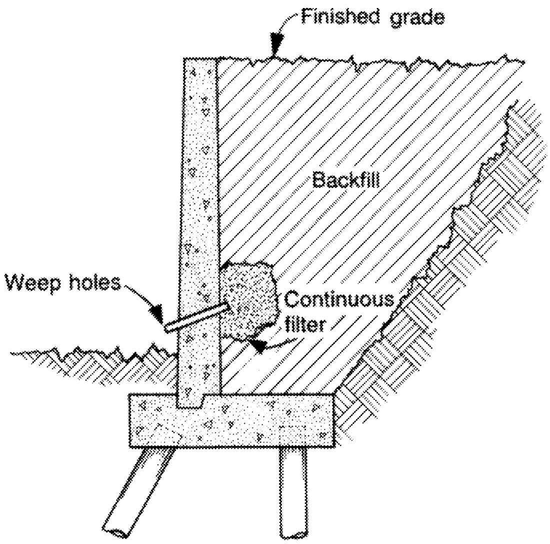


FIGURE 10-19. Retaining wall with battered piles.

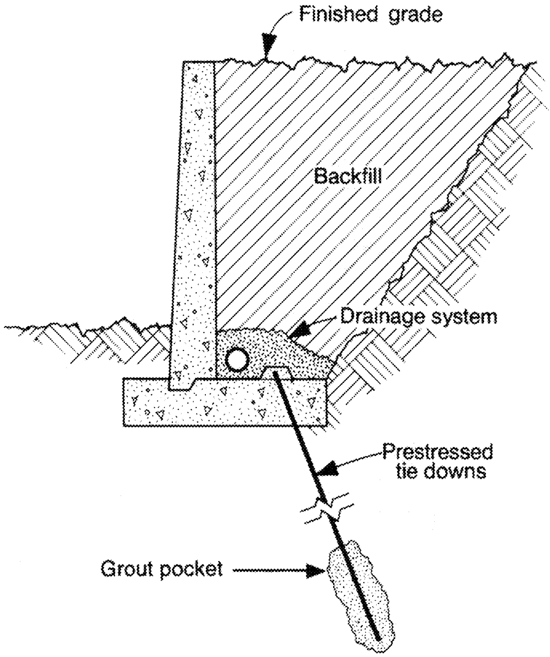


FIGURE 10-20. Retaining wall with prestressed tiedowns.

dense and hard. A typical tiedown arrangement is illustrated in Figure 10-20. Note that because the tiedown is sloped it will provide stability to the wall through both vertical and horizontal components:

1. The vertical component increases the resistance to rotation and adds to the frictional resistance at the base of the footing by increasing the pressure of contact.
2. The horizontal component directly increases the resistance to slide.

Tiebacks

The third system of providing additional lateral earth resistance to a wall is to anchor the wall back into undisturbed earth with prestressed tiebacks. This eliminates most or all of the horizontal thrust from the footing. Tiebacks are much more versatile than tiedowns and may be used for any of the following purposes:

1. To provide lateral support to a temporary retaining wall constructed as an integral part of a major excavation process, as previously illustrated in Figures 10-3 and 10-5. These cables provide not only direct horizontal support but also increase the vertical seating pressure at the base of the wall.
2. To provide lateral support to a permanent retaining wall by tying the wall back into the undisturbed soil at some distance behind the backfill as illustrated in Figure 10-21.

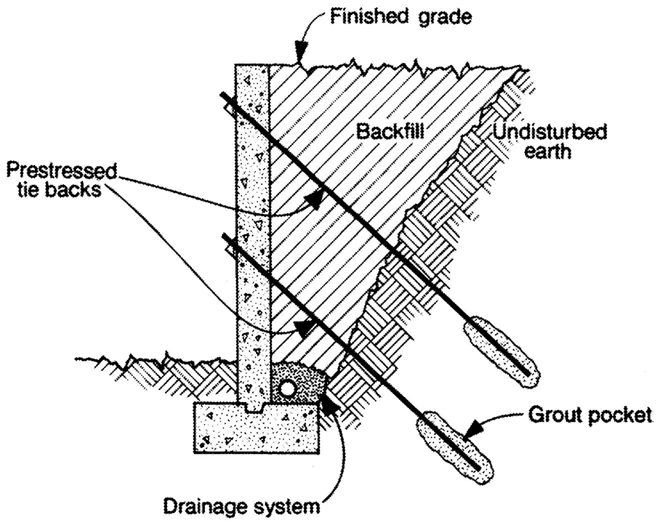


FIGURE 10-21. Retaining wall with prestressed tiebacks.

Tiebacks, of course, can only be used when the owner of the retaining wall has legal access to the adjacent underground property.

Design Differences Between Tiedowns and Tiebacks

Although the tiedown system of Figure 10-20 and the tieback system of Figure 10-21 produce the same result, it is of interest to note the major differences between them. The tiedown system is that of a true cantilever retaining wall, whereas the tieback system is that of a wall designed to span vertically between anchor points. The tiedown system will therefore require heavier reinforcing, a wider footing and considerably more excavation. It should also be noted that the cables in the tieback system are more difficult to install and consequently more expensive.

A Word of Caution

Tiedown and tieback systems can offer a relatively easy and cost effective way of providing a substantial anchorage force. This is true, however, only when the exposed earth walls of the bore hole have sufficient cohesiveness to remain stable until the ties have been installed and grouted. If these walls will not stand, as in the case of granular soil, it may be possible to mechanically stabilize them. This may be accomplished by injecting a bentonite slurry during the drilling process or by installing a temporary steel liner. These methods were previously described in Article 8-7. The installation of a tieback system in granular soil will be particularly difficult to control because of the elevated angle of the bore hole.

It is very important that the characteristics of the soil be determined during the working drawing stage of the project so that the work as specified, can be built. The advice of a general contractor with experience in these matters can prove to be invaluable.

10-12. PRESTRESSED TIEDOWNS AND TIEBACKS

Purpose

Tiedowns and tiebacks are high strength cables that are prestressed and post-tensioned. The cable and anchorage assembly is frequently called a tendon. These cables are used to apply an external force that will provide additional restraint against lateral pressure or rotation of the structure. These cables are anchored into bedrock or satisfactory soil with a pocket of grout. This pocket transfers the force from the tendon into the supporting rock or soil through friction. The end of the cable at the grout pocket is called the dead end anchorage. The other end, where it is attached to the structure, is called the live end anchorage. Although not necessarily anchored into bedrock, the grout pocket is frequently referred to as a rock anchor. When anchored into soil, the pocket should more appropriately be called a soil anchor, or simply a grout pocket. At the live end anchorage the cables are anchored to the structure by wedges and bearing plates so as to transfer their tensile force directly into the structure.

Cables must be anchored into rock or very stiff, dense soil in order to provide satisfactory anchorage. If such anchorage is not available, this system of lateral restraint cannot be used. Cable anchorage into soil must be treated with caution. The problems associated with anchorage into soil are twofold:

1. The soil must have sufficient strength to develop the required holding power in a grout pocket not exceeding approximately five feet in length. In order to satisfy this requirement the soil should be classified as very stiff or hard clay, having an unconfined compression strength of at least 3.6 tsf, resulting in a cohesive strength of at least 3600 psf. Even with this strength it will generally be found that the full tensile capacity of the cable cannot be developed by the grout to soil bond. This may result in substantially reducing the allowable tension in the cable.
2. The soil must have sufficient cohesiveness that the side walls of the bore hole will stand intact until the operation has been completed. If a bentonite slurry would be required in order to stabilize the side walls then the alowable bond stress would have to be reduced to a level where it would no longer be practical to use this method.

Material

The cable used in this type of prestressing work usually consists of single or multiple seven wire strands. Each strand is a shop fabrication of six wires spun helically around a straight, central wire. The wires are drawn from high strength steel having an ultimate strength in the range of 250 to 270 ksi. The effective prestress which can be developed by a single strand varies between 5.4 to 24.6 kips, depending upon the area of the strand and the ultimate strength of the wire. These are the same cables used in post-tensioned building construction. Design of these elements should follow the guidelines set forth in the ACI Building Code.

Anchorage

Cable installation must satisfy 2 criteria: adequate anchorage at the grout pocket, and positive protection for the cable above the grout pocket. Cable anchorage is described herein. Protection is described in a later paragraph.

The first thing that must be done is to drill the holes. These holes are usually four inches in diameter but may be larger when multiple strands are used. A typical tiedown drilling operation is shown in Figure 10-22. Note that the reader

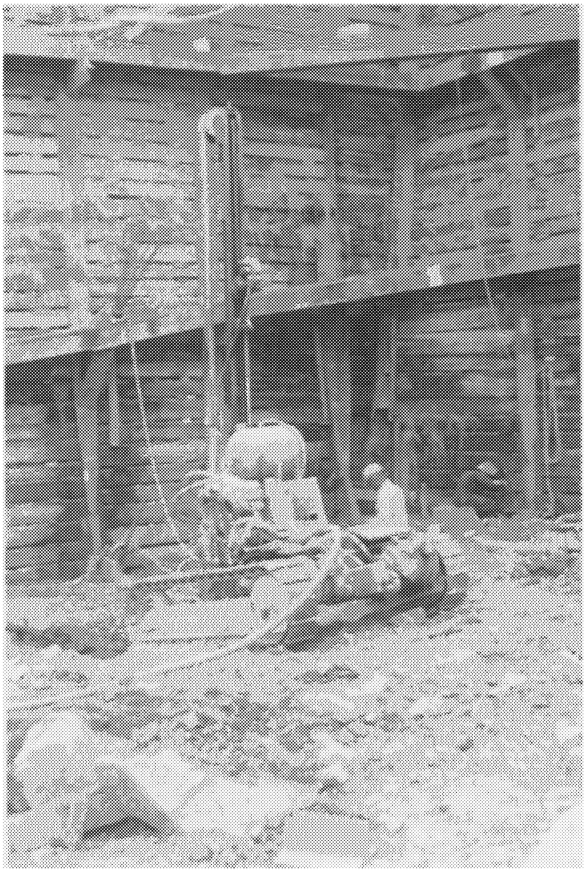


FIGURE 10-22. Rock anchor hole being drilled. [Ref. 9]

FIGURE 10-23. Rock anchor cable and grout tube positioned in hole prior to grouting the dead end anchorage. [Ref 9]

can plainly see the soldier beams and timber lagging which were used during excavation of the site. The anchorage end of one of the wall tiebacks is also visible.

In order to fully develop the cable, the cable must be adequately anchored into the ground at the dead end anchorage. The length of anchorage should be limited to approximately 5 feet. If this length will not develop the full strength of the cable, the strength should be reduced. Immediately before anchoring the cable, the hole should be cleaned out with compressed air or by flushing with water. The cable is then lowered into the hole to within about 4 inches of the bottom. A one inch diameter flexible grout tube is also lowered into the hole, and a special formula nonshrinking cement grout is pumped down into the area required for the grout pocket. This installation just before grouting the tiedown cable is shown in Figure 10-23.

After the grout has reached its required strength, the cable is tensioned at the live end anchorage with hydraulic jacks. Proper cable tension is determined by jacking force and cable elongation. After the cable is properly tensioned, it is secured with steel wedges that transfer the force from the cable into a bearing plate assembly that bears directly onto the concrete. Cable elongation is measured as shown in Figure 10-24.

Transfer of Tendon Force

The transfer of force at the dead end anchorage requires two steps beginning with the transfer from cable to grout and ending with the transfer from grout to

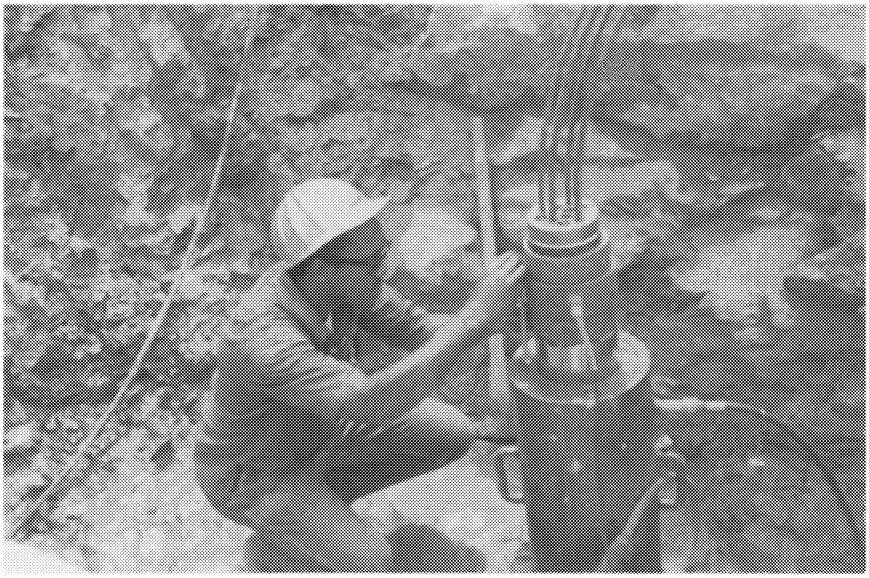


FIGURE 10-24. Measuring elongation after the cable has been prestressed. [Ref. 9]

rock or to soil. It is essential that a premixed, nonshrinking grout be specified. This should be a graded sand–cement mixture having a maximum water/cement ratio of 0.35 to 0.40. The following values may then be used for preliminary design:

1. For transfer from cable to grout: The ultimate bond strength at failure may be taken as:

   2.0 to 5.5 MPa = ± 300 to 800 psi [Ref. 12]

   A safety factor must be applied to this ultimate strength.
2. For transfer from grout to rock: The allowable bond strength at working stress may be taken as:

   0.35 to 1.40 MPa = ± 50 to 200 psi [Ref. 12]

3. The transfer from grout to soil: The allowable bond strength at working stress may be taken as 12.5 psi. This stress originated from data given in Article 8-10 wherein the allowable shear stress between a drilled pier and a clay soil was limited to one-half the cohesion or 1800 psf, whichever was lesser. It should again be noted that soil anchors will generally not develop the full tensile capacity of the cable. The design strength in the cable, therefore, may have to substantially reduced.

Note:

1. These bond strengths shall only be used to provide the basis for the base bid. All strengths must be confirmed by field tests.
2. In each instance the transfer force will be the product of the permissible bond strength times the perimeter of contact times the length of contact. In no case shall the allowable force in the cable be exceeded.

Destruction Tests

The ultimate strength of the transfer of forces at the dead end anchorage can only be accurately determined by testing a completed cable assembly to destruction.

The test is started by applying an initial seating force of approximately one-half of the computed ultimate strength of the cable. The force is then increased incrementally until literal failure takes place. It is desirable that failure should occur in the cable and not at the dead end anchorage. Therefore, if failure does occur at the dead end anchorage the length of the grout pocket is increased and further tests performed. After it has been determined that the ultimate strength of the cable is the governing factor, the design load can then be confidently determined by applying an appropriate safety factor, usually taken as 2 or 2½.

It should be noted that all work regarding the destruction tests should proceed expeditiously because no permanent work can be installed until the tests have been successfully concluded.

Because these are destruction tests, they cannot be performed on cables that are intended to be a part of the permanent work.

The number or destruction tests required depend on the site and on the size of the project. Two tests are considered to be a reasonable minimum. When tests borings indicate variations in the rock or soil characteristics throughout the site more tests would be required. This is another of the many reasons why test borings are so very important to the success of those engaged in the design and construction of the project.

Staging of Prestress

Tiedowns are elements that basically provide vertical resistance. Because of their location with respect to the footing, it is necessary that these elements be stressed to their final design strength prior to the wall being backfilled.

Tiebacks are elements that basically provide horizontal resistance. It is the pull of the tiebacks and the push of the backfill that stabilizes the wall. If either operation were to proceed without the balancing effect of the other, this would force the wall out of alignment.Were this to happen, the wall would quite probably sustain serious damage. The stressing of the tiebacks must therefore be carefully staged with the installation and compaction of the backfill. Staging must be carefully coordinated between the engineer and contractor.

Protection of Cables

The length of cable contained in the grout pocket is called the fixed length. The length beyond the grout pocket is called the free length. All cables considered to be part of a permanent installation must be adequately protected throughout their full length against the adverse, long term action of soil, water, and air. To provide adequate protection, the free length of the cable must be encased with the same nonshrinking grout that is used to set the grout pocket. There are two procedures by which this can be accomplished: one requires that the grout be poured in two stages; the other requires only one stage. These procedures are described as follows.

Two-Stage Procedure. The cable is lowered into the bore hole to within approximately 4 inches of the bottom. The specified length of grout pocket is then carefully filled with grout, making sure not to overfill, as this would let grout push up into the free length. After the grout has developed its full strength, the cable is jacked and secured, after which the free length of the cable is grouted.

One-Stage Procedure. The free length of the cable is encased in a greased plastic sheath in which the cable can move freely. Care must be taken that the sheath does not encroach into the grout pocket, because this part of the cable must remain bare in order to bond with the grout. With this arrangement, the full length of the cable can be grouted in one operation. After the grout has developed its full strength, the cable can be jacked and secured.

The reason why the entire length of cable cannot be grouted in one pour without sheathing the free length is because the cable will noticeably elongate due to the relatively high level of applied stress. This elongation may crack the grout, thereby subjecting the cable to possible erosion from contaminants in the soil or water.

This is also the reason why the length of the grout pocket is limited to approximately 5 feet.

Although both procedures have a record of successful performance, there are several reasons why the two-stage procedure may be preferred, even though more expensive. In the two-stage procedure, the free end of the cable is held perfectly straight because it is under tension. The sheathed cable, on the other hand is free to wobble. With the sheathed cable, there is a plane of weakness where the grout thickness is abruptly changed at the base of the sheath. This is a potential source of cracking and influx of water. Grout pockets are occasionally installed under the water table. The sheathed cable procedure should not be used in situations where the sheath would be immersed in water.

Alternate Design of Anchorage

When using the installation procedures previously described the tension in the cable is transferred into the grout through perimeter bond. This induces tension of varying intensities throughout the length of the grout. With grout pocket lengths of approximately five feet experience indicates that the incidence of cracking will not adversely affect the performance of the assembly through erosion.

An alternate to this procedure would be to fit the end of the cable with a bearing plate approximately 1 inch less in diameter than the bore hole. Transfer would then be through compression rather than by tension. Using this alternate, both the free length and the fixed length of the cable should be sheathed. The advantage of this alternate is that the entire length of cable acts as a gout pocket, thereby substantially increasing the transfer area between grout and soil. A disadvantage of this alternate is that the bearing area is rather small and may not fully develop the tensile capacity of the cable. Increasing the size of the bore hole would add considerably to the bearing area and thus to the overall strength of the system. A larger bore hole would, however, add substantially to the cost.

10-13. REQUIREMENTS RELATIVE TO BACKFILL

Material Source

There are two sources from which bulk earth can be obtained for use as backfill:

1. The earth removed during excavation represents a potential source of material for use as backfill, subject to suitability. This material is sometimes referred to as residual earth or earth spoils.
2. When the material at the site is unsuitable, earth that is suitable must then be brought to the site from another location. This earth is usually referred to as borrow fill.

Factors in the Selection and Use of Backfill

The architect and engineer must consider several factors when they write the specifications regarding the selection and use of the material to be used for backfill. These considerations include:

1. All soil use as backfill shall be completely free of organic material. Refer to Article 1-9 for tests that will indicate the presence of any organic material.
2. The material must be available for delivery in the quantity required for the work to proceed without delay or interruption.
3. The material must be cost effective. This is not to say that unsuitable materials must be used just because they are less expensive. Some materials, however, may perform almost as well and may cost considerably less.
4. The material must be such that it can be compacted to the density required for its intended use, as specified in Table 12-1.

5. The backfill must exhibit sufficient strength characteristics to support its own weight and the weight of any surcharge without undue settlement.
6. The material must have sufficient inherent permeability to insure adequate drainage of rain water behind the wall. A discussion of permeability is given in a later paragraph.
7. Soils with medium to high levels of shrink-swell potential, defined as soils having a plasticity index greater than ten, should never be used as backfill. This is one instance in which borrow fill is mandatory. The inclusion of hydrated lime or portland cement into the shrink-swell soil as a means of stabilization is considered unacceptable because of the continuous availability of water due to rain or runoff. Refer to Article 13-6 for further information on using the plasticity index as an indication of swelling potential.
8. Backfill should be free of all extraneous materials such as roots or tree stumps or construction spoils such as formwork, building paper, or any other material that would eventually rot away and cause a change in soil volume.
9. Backfill must be cleared of rocks, bricks, pieces of stone, bolts, nails or any other hard, sharp material that could damage waterproofing, drainage systems, or underground mechanical services.
10. Backfill should not be placed on frozen ground nor should it contain any frozen material because of the enormous damage which can occur when the material thaws.
11. Backfill should be mixed and deposited in such a way as to produce reasonable uniformity throughout the mass.

Regardless of the material selected for backfill, field inspection combined with tests both in the field and in the laboratory must be performed to insure that the backfill meets specification both as to material and installation. Refer to a later paragraph at the end of this article relative to design responsibility.

Use of Granular Materials

There is general agreement among design professionals that granular soils such as sand, gravel and crushed stone are the preferred materials for use as backfill. The reasons for this are as follows:

1. Granular materials can be bulk mixed to produce a uniform, well-graded mass.
2. A backfill consisting solely of granular material exerts less lateral pressure than any other kind of soil.
3. Soils consisting solely of granular materials are inherently self-draining. Rain water and surface water runoff can readily percolate through the backfill and enter into the drainage system beneath.

4. A granular mixture is relatively easy to compact to the specified density and is not subject to over compaction.
5. The physical and chemical properties of a granular material are not adversely affected by the action of water.

Those soils conforming to designations GW, GP, GM, and SW of the Unified Soil Classification System meet all the above characteristics and are considered as good to very good materials for use as backfill in those situations where compaction, bearing capacity and settlement characteristics are of primary importance.

Use of Cohesive Materials

Cohesive soils, particularly those consisting predominately of clay, are generally considered to be undesirable for use as backfill. The reasons for this are as follows:

1. It is relatively easy to overcompact clay while the wall is being backfilled. The effect of overcompacting is to increase the lateral pressure beyond that which the wall was designed to resist. The safety factors against overturning and sliding are therefore reduced and failure may occur. There are instances of walls having failed during construction because of overcompaction of backfill.
2. All clays have a tendency to shrink or swell. Clays having a high swelling potential, previously defined as soils having a plasticity index greater than ten, should never be used as backfill since these clays can exert very large lateral pressures when restrained against free expansion.
3. Clay deposits are relatively impermeable which makes adequate drainage in back of the wall impossible. The wall must therefore be designed to resist water pressure in addition to the pressure of the earth backfill. This is uneconomical and may be needlessly wasteful.
4. The physical properties of a cohesive soil may be significantly altered under the action of water.

Permeability

Permeability is a term used to indicate the potential for movement of water through the voids within a soil mass. In order for movement to occur the voids must be continuous throughout the mass. The flow of water within the soil voids is a function of the size of the voids, their shape, and general characteristics.

Because of the irregularity of void spaces within a soil mass the best procedure by which the movement of water can be predicted with reasonable accuracy is to run field or laboratory tests on the soil in question.

The coefficient of permeability, symbolized by (k), may be used to estimate the rate at which water will permeate through the mass. This coefficient is defined in the following formula:

$$k = 100 \times D_{10}^2$$

TABLE 10-1. Coefficient of Permeability (k) [Ref. 18]

Permeability	*k*-cm/sec	Type of Soil
Very high	$>10^0$	Clean gravel
High	$10^0–10^{-1}$	Gravel, clean coarse sand
Medium	$10^{-1}–10^{-3}$	Graded sand, fine sand
Low	$10^{-3}–10^{-5}$	Silty sand, silt
Very low	$10^{-5}–10^{-7}$	Dense silt, clayey silt
Impermeable	$<10^{-7}$	Clay, silty clay, bedrock

In this formula the coefficient of permeability is expressed in centimeters per second. D_{10} is the particle size taken from the particle distribution curve at the 10% passing mark. In this formula, D_{10} must be expressed in centimeters. k values and drainage characteristics of various soils are given in Table 10-1.

As an example of the use of this formula assume a sandy soil having a grain size of 0.3 mm at D_{10}; therefore:

$$k = 100 \times 0.03^2 = 0.090 \text{ cm/sec}$$

This coefficient, according to Table 10-1, indicates that the soil can be expected to exhibit a medium permeability.

Installation of Backfill

Backfill must be carefully deposited into the excavation and mechanically compacted to form a uniform, dense, and stable mass. Proper compaction is necessary for the following reasons:

1. To provide a predictable stability within the mass
2. To control future settlement of the backfill, the adjacent earth and any structures or utilities in close proximity to the wall
3. To prevent the influx of deleterious materials into the backfill due to rain, surface water runoff, or seepage from the adjacent ground

Backfill should be placed and compacted in successive layers not exceeding 8 to 12 inches in thickness. Each layer should be mechanically compacted with hand-held, machine-driven tampers. Compaction by the use of flooding or by any other kind of water infiltration should not be permitted.

Requirements relative to compaction are discussed in detail in Chapter 12.

Design Responsibility

The successful long term performance of any basement or free standing retaining wall depends on the proper selection of material to be used as backfill and on the proper installation of that material. During the design stage of the project, the architect and engineer should consult with a general contractor and an excavator, both of whom have expressed an interest in submitting a bid on the project. The purpose of these consultations is to determine what kind of material is readily available and whether there are sufficient quantities with which to complete the job. Selection of material is based on the following criteria:

1. Only soil readily identified as conforming to one of the USCS soil groups itemized in Article 12-2 shall be considered. The compaction characteristics of these groups are given in Table 12-3.
2. The design team must determine the minimum relative density to which this backfill must be compacted. This is a function of the location and usage of the backfill. Guidelines for this purpose are given in Table 12-1.

After the soil classification has been established, the engineer can determine the lateral pressure for which the foundation walls must be designed. This design will then become a part of the contract documents.

The architect and engineer do not normally provide an in-depth inspection of the work in progress, and for that reason they are not normally responsible for overseeing the actual installation of the backfill. On major construction the owner will engage a qualified inspection agency or testing laboratory to perform the in-depth inspection and such soils testing as required by contract with the architect and engineer.

10-14. DRAINAGE

General

When there is a permanent water table behind the wall, drainage by any means would be ineffective and possibly even detrimental to the area. The wall must then be designed to resist the resulting water pressure. When there is only incidental ground water behind the wall, drainage then becomes an extremely important design consideration. As previously noted, the purpose of drainage is to prevent the buildup of incidental water pressure. Such a buildup could occur because of rainstorm, snow melt, surface runoff, or seepage of water from the adjacent ground. In order to eliminate the possibility of incidental water buildup, a suitable, permanent drainage system must be installed at the base of the wall prior to installation of the backfill. Details relative to this drainage system are illustrated in the following paragraphs.

Drainage System—Basement Wall

A typical drainage system used in basement wall construction is illustrated in Figure 10-8. For this system, a perforated pipe no less than six inches in diameter is installed at the base of the wall and is extended for the full length of the wall. The pipe should be plastic rather than iron or steel in order to avoid future problems with rust. The joints of the pipe should interlock in order to avoid future problems with clogging of the pipe due to the influx of fines through an open joint. When the joints cannot be of the interlocking type, they should be fitted tight and secured with a double wrapping of galvanized wire mesh similar to window screening. For proper performance the pipe should be laid with perforations faced downward in order to impede the influx of fines into the pipe.

The pipe should be pitched at no less than one-quarter of an inch per foot and should discharge by gravity flow into one of the following types of permanent collector systems:

1. A storm sewer system
2. A system of additional piping that will disperse the water onto a ground surface situated below and away from the building
3. A sump located within the building from which collected water can be pumped to a permanent disposal area away from the site

The pipe must be completely surrounded by a thick envelope of selected sand and gravel called a drainage filter whose purpose and design is discussed in a later paragraph. Care must be taken during construction to ensure that this envelope does not undercut the footings. In the event of unintentional undercut, all such areas should be filled with 2500 psi concrete.

The surface of the ground should be graded away from the building at no less than one-quarter of an inch per foot.

In no case should a drainage system be considered as negating the need to dampproof or waterproof the basement perimeter.

Drainage System—Cantilever and Gravity Retaining Walls

If at all possible, drainage for free standing retaining walls should follow the general guidelines for the drainage of basement walls. This necessitates the installation of a continuous collector system, as indicated in Figures 10-20 and 10-21.

In those instances when permanent collector systems are not available, one of the following alternates must be adopted:

1. Extend the collection pipe underground to some natural disposal. This could be a lake, a stream or an open drainage ditch. This system must rely solely on gravity.
2. Install a system of through-the-wall weep holes. This system of drainage is indicated in Figures 10-18 and 10-19.

The weep hole system uses the same kind of continuous drainage filter in back of the wall, but without the perforated pipe. Water collected in the drainage filter discharges to the front side of the wall by draining through a series of holes called weep holes. These holes should be placed no farther than 4 to 6 feet apart and must be installed throughout the full length of the wall. The holes are usually lined with plastic pipe, which is set into the forms before the wall is concreted.

The pipes should be at least 4 inches in diameter and should be pitched 2 inches toward the front of the wall. The end of the pipe contained within the backfill should be covered with rust proof wire mesh and encased in an envelope of pea gravel.

The weep hole method is very popular with contractors due to its relative ease of construction and low cost. It is not, however, as positive a system of drainage as the one in which the collected water is discharged into a permanent disposal area.

Drainage Filter Design

The purpose of a sand and gravel drainage filter is to facilitate the entrance of drainage water into the pipe and to prevent the perforations in the pipe from being clogged by fines which may be carried by the water. The materials which make up the filter must be carefully graded so as to permit the unimpeded flow of drainage water. Also, the void ratio and the size of the voids must be carefully controlled to prevent the filter itself from being contaminated with fines. The design of the filter system is usually assigned to the soils engineer, whose design is subject to the following general requirements relative to the ratio between the particle sizes in the filter material and those in the backfill:

1. In order to insure that water will flow freely into the drain pipe, the permeability of the filter material must be considerably greater than the permeability of the soil being drained. To satisfy this requirement, the following ratio should be used:

$$\frac{D_{15} \text{ of filter material}}{D_{15} \text{ of backfill}} > 4 \text{ [Ref. 13]} \tag{10-1}$$

2. The filter material must be carefully graded so that fines from the soil being drained will be prevented from migrating into the filter system and eventually clogging it. The following ratios should be used:

$$\frac{D_{15} \text{ of filter material}}{D_{85} \text{ of backfill}} < 5 \text{ [Ref. 13]} \tag{10-2}$$

$$\frac{D_{50} \text{ of filter material}}{D_{50} \text{ of backfill}} < 25 \text{ [Ref. 13]} \tag{10-3}$$

$$\frac{D_{15} \text{ of filter material}}{D_{15} \text{ of backfill}} < 20 \text{ [Ref. 13]} \tag{10-4}$$

3. And, finally, to insure that the particles comprising the filter material will not clog the perforations in the pipe, the following requirement should be met:

$$\frac{D_{85} \text{ of filter material}}{\text{diameter of opening}} > 2 \text{ [Ref. 10]} \tag{10-5}$$

The D designations used in the above criteria refer to particle dimensions obtained during a sieve analysis performed in the laboratory. D_{15}, for example, identifies a specific size of particle for which 15% of the soil, when measured by weight, is of smaller size.

In a standard sieve analysis the opening through which the particle must pass is square. The particle, however, is not square and will have different dimensions of length, breadth, and height. The size referred to by the D designations, therefore, does not in all probability represent the largest dimension nor the smallest dimension of the particle but will represent some dimension in between. The smallest sieve through which the particle will pass is determined by the alignment of the particle with respect to the opening in the sieve.

10-15. SAMPLE PROBLEMS

Example 10-1

Required:

First: To classify the soil proposed for use as backfill for a cantilever retaining wall and, particularly, to determine whether it is poorly-graded or well-graded.

Second: To determine whether the soil proposed for use in the continuous filter system satisfies the requirements of a good filter material.

Given: The sieve analysis of each material is given in Table 10-2.

The material proposed for use as backfill will be classified according to the United Soil Classification System, as described in Article 1-9.

1. The material is first classified as coarse grained. This is because more than 50% (actually 96%) of the dry weight of the sample is retained on a No. 200 sieve.
2. The material is next classified as sand because a greater percentage of the coarse fraction (actually 78% of 96%) passes a No. 4 sieve.
3. It is also noted that less than 5% of the sample (actually 4%) passes a No. 200 sieve.

For the condition noted in item 3, the flow chart in Figure 1-8 requires that the coefficients of uniformity and curvature be evaluated in order to classify the

TABLE 10-2. Example 10-1—Percentage of Total Weight Passing[a]

Sieve Size	3″	¾″	#4	#10	#40	#200
Backfill	100%	100	78	54	20	4[b]
Filter	100%	76	34	12	2	0[b]

[a] Percentages are based on dry weight.

[b] These values are plotted in Figure 10-25.

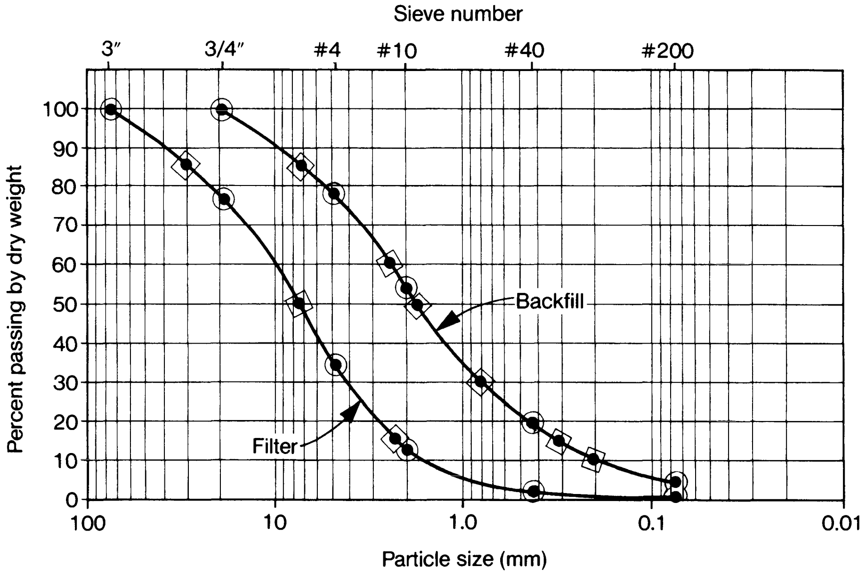


FIGURE 10-25. Points shown circled are plotted from Table 10-2, points shown boxed are read off of the curve and recorded in Table 10-3.

sample as well graded or poorly graded. The use of these coefficients is described in Article 1-5.

In order to do this work, it is necessary to draw a particle distribution curve of the backfill material. Because later work requires similar data regarding the filter material, both curves have been shown in Figure 10-25.

The particle size corresponding to the required D values is read off of each curve. The subscripts of the D values represent the vertical point on the curve at which the percentage of particles, by weight, is finer. The numbers thus obtained are given in Table 10-3.

To determine the grading characteristics of the backfill, compute:

$$C_u = \frac{2.4}{0.20} = 12 \qquad \text{and} \qquad C_c = \frac{(0.80)^2}{2.4 \times 0.20} = 1.3$$

For well-graded sand, the coefficients must satisfy the following:

$$C_u > 6 \qquad \text{and} \qquad 3 > C_c > 1$$

TABLE 10-3. Example 10-1—Particle Size in Millimeters[a]

Percentage	D_{10}	D_{15}	D_{30}	D_{50}	D_{60}	D_{85}
Backfill	0.20	0.31	0.80	1.7	2.4	7.2
Filter	—	2.3	—	7.4	—	30

[a]These sizes are read off of the curve in Figure 10-25.

It can be seen that the backfill satisfies these requirements and can finally be classified as a well-graded sand.

To determine the suitibility of the filter material, it is first noted that the particle distribution curve of this material more or less parallels that of the backfill. This is generally indicative of a well chosen filter material.

Calculations for filter adequacy are as follows:

$$\text{From Formula (10-1): } \frac{2.3}{0.31} = 7.4 > 4 \text{ ok}$$

$$\text{From Formula (10-2): } \frac{2.3}{7.2} = 0.32 < 5 \text{ ok}$$

$$\text{From Formula (10-3): } \frac{7.4}{1.7} = 4.4 < 25 \text{ ok}$$

$$\text{From Formula (10-4): } \frac{2.3}{0.31} = 7.4 < 20 \text{ ok}$$

The filter material, therefore, satisfies all requirements.

11

Walls—Design Considerations

11-1. LATERAL PRESSURE DESIGN REQUIREMENTS

General

The purpose of a retaining wall is to retain earth. In order to properly design the wall, it is necessary for the designer to determine, with reasonable accuracy, the magnitude of the lateral pressure to which the wall will be subjected. This requires an understanding of the physical properties and in-place characteristics of the earth which the wall is to support. The importance of this understanding cannot be overemphasized.

The lateral pressure to which a wall may be subjected depends on the following items:

1. The anticipated procedures regarding excavation, as dependent on the characteristics of the existing earth and the preference of the contractor
2. The characteristics of the backfill and the density to which it is to be compacted
3. Is the surface of the backfill level or does it slope?
4. The existence of a water table
5. The presence of surcharge or the possibility of future surcharge.

Numerical values representative of the lateral pressure produced by these items is illustrated in the pressure diagrams and charts of Chapter 9.

The Effect of Excavation and Backfill

Before construction of the wall, the immediate area must be excavated to provide a clear, safe place in which to work. After the wall and its supporting elements have been completed, the excavation must be backfilled. The lateral pressure to which the wall will be subjected will be determined by the characteristics of the backfill or of the undisturbed earth beyond the excavation, depending on the following:

As discussed in Article 9-2, no excavation in earth will stand permanently with vertical, or near vertical side walls. Soils with little or no cohesion will collapse as they are being dug. Soils with noticable cohesion will stand for some period of time, but they too, will ultimately fail. The author has seen this happen on numerous occasions, particularly while test pits were being dug during a preliminary site investigation.

Excavation in rock, of course, is an entirely different matter.

There are three different situations relating to excavation that should be explored for the purpose of determining their effect on the source of the lateral pressure for which the wall must be designed. These are illustrated in Figure 11-1.

Figure 11-1(a) shows excavation in sound rock. This kind of excavation requires considerable drilling and blasting, after which the broken rock is loaded and removed by hand and machine. The exposed side wall of the excavation is inherently stable and will not collapse. The backfill in this instance will be the source of the lateral pressure for which the wall must be designed.

Figure 11-1(b) shows excavation in cohesive soil. When excavating in stiff, cohesive soil, it may be determined that the side walls will remain in tact for the duration of the construction process. A narrow width of excavation may be possible. In this instance, the wall must be designed for the lateral pressure which will ultimately be exerted by the existing earth because the pressure of that earth will be transmitted to the wall through the narrow width of backfill.

Figure 11-1(c) shows excavation in granular soil. When excavating in any kind of granular soil or in soft to medium clayey soil the side walls of the excavation will not stand. The excavation must therefore remove sufficient earth so that the surface of the cut approximates the angle of repose of the soil. In

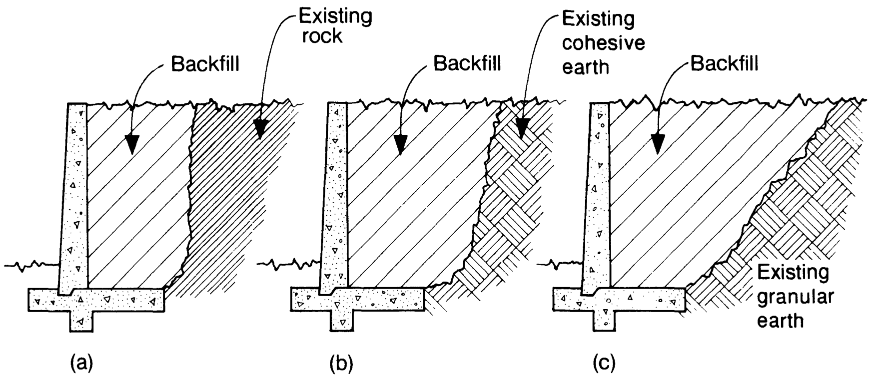


FIGURE 11-1. Excavation profiles for different types of soil.

this instance, the wall should be designed to resist the pressure induced by the backfill. A similar condition can also occur even with a truly cohesive soil whenever the contractor elects to make an excavation having a relatively gentle slope.

Figure 11-2 is an example of an excavation in a mixed grained soil whose characteristics vary with depth. Note that the lower part of the soil has sufficient cohesion that a relatively steep embankment can be excavated and maintained for a short period. This is indicative of the type of excavation shown in Figure 11-1(b). The upper part of the soil, however, exhibits more granular characteristics, thus requiring that the degree of slope be lessened, to approximate the condition shown in Figure 11-1(c).

When excavating in earth, the contractor would normally prefer to remove as little earth as possible, consistent with the requirements of construction clearances. The narrow excavation shown in Figure 11-1(b) would, therefore, normally be preferred by the contractor, and he may choose to use it as long as the side wall of the earth remains stable. If this earth starts to spaul off, the contractor has two options:

1. Reduce the slope of the excavation thereby requiring the removal of more earth, or
2. Shoring the excavation until the work is complete.

The ability of the earth to stand without spauling off can be approximated from information obtained from the borings. The sides of stiff, cohesive soils

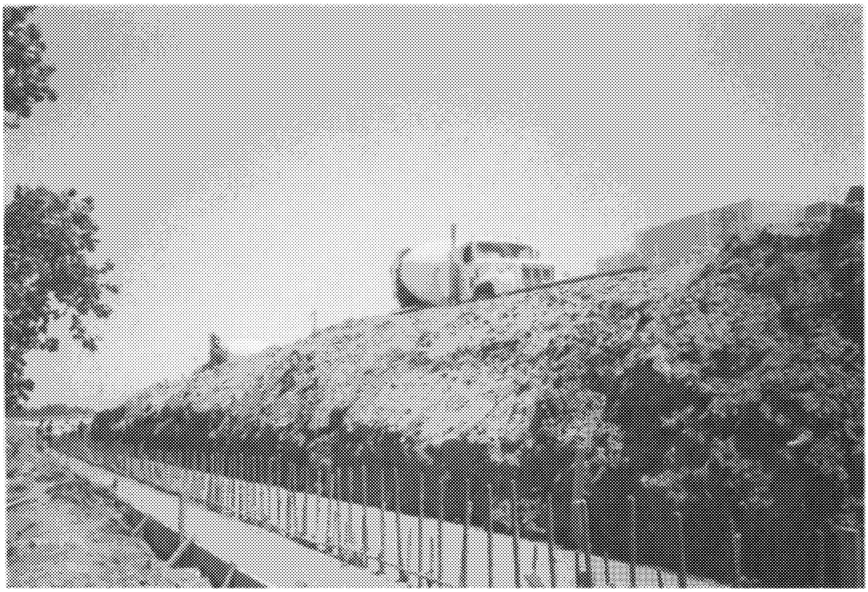


FIGURE 11-2. Soil variation within the depth of an excavation.

will stand for a reasonably long period of time. The contractor, however, must be aware that adverse weather, such as heavy rainfall or freeze, can cause sudden failure of the embankment. The stability of clay embankments is also a function of moisture content. Embankments that dry out are especially susceptible to failure. This can be of serious concern, particularly in hot weather, because of the extent of the exposure of the side walls to atmospheric conditions.

In determining the lateral pressure for which the retaining wall must be designed, prudent engineering calls for a conservative approach. It is recommended, therefore, that the lateral pressure characteristics of both the backfill and the existing earth be determined and that the larger of the two be used for design.

For a detailed discussion regarding the determination of the lateral pressure characteristics for different soils, refer to Chapter 9.

Concerns Regarding Water Pressure

Water can enter the backfill situated behind a retaining wall from either or both of the following sources:

1. There is a point below the surface of the ground beneath which the ground is saturated with water. The elevation of this point is known as the ground water table or simply as the water table. The water occurring below this point is known as ground water. This is the water that one would find when digging a well. Beneath this level all the voids and pores within the soil are completely filled with water. Owing to a phenomenon known as capillary action, it is likely that voids somewhat above the water table will also be saturated or in some instances partially saturated. The water table can be dramatically affected by long-term changes in climate. Extended periods of drought will lower the water table, just as extended periods of precipitation will raise it. The water table is not nearly as affected by short-term changes in weather.
2. Rain water and surface water runoff can flow directly into the backfill or can seep into it from the adjacent ground.

When the ground water table is below the base of the wall, and no rise in the table can reasonably be expected, the wall need not be designed for water pressure provided that a suitable drainage system is installed at the base of the wall. The purpose of this system is to prevent a build up of water behind the wall due to an influx of rain water, surface water runoff or seepage. The drainage system must be adequately sized and should depend solely on gravity to provide a positive uninterrupted escape of water from behind the wall. In those cases when the soil behind the wall exhibits poor drainage, a build up of water should be anticipated. It is incumbent upon the designer to design the wall to resist the anticipated water pressure in addition to the pressure exerted by the earth.

When information obtained from test borings or from other sources indicates that the water table is presently above the base of the retaining wall or may rise above it at some time in the future, the wall must then be designed to resist water pressure in addition to the pressure exerted by the earth. When test borings are taken during a sustained period of dry weather or drought, consideration must be given to the possible rise in the water table once conditions return to normal.

There are procedures whereby the water table can be lowered in the immediate vicinity of the wall. Because of the possibility that the water table may extend well beyond the construction site, such drawing down of the table should be considered only as a temporary measure for the benefit of the construction process. It should be noted that during construction ground water will continuously seep into the excavation. This seepage must be adequately controlled so as not to hinder construction. After construction has been completed and the area backfilled, ground water will then seep unimpeded into the backfill from the adjacent ground.

Lateral Load Due to Surcharge

Surcharge is defined as the weight of any structure or physical thing that would cause the earth beneath it to exert a lateral thrust on the wall under consideration. It can be seen that the location of the surcharge is very important because the closer it is the more effect it will have. The designer must consider not only the surcharge which presently occupies the adjacent ground but that which may be built at a later date. The designer, therefore, must consider the future effect of zoning.

11-2. BASEMENT WALL DESIGN OPTIONS

The earth pressures resisted by the basement walls must be transferred into other building elements having the strength and stability to provide the lateral support necessary to achieve static equilibrium. The wall must be designed to span between these supports and to impart the earth pressure to them. Subject to the way in which the building elements are laid out, the wall may be designed to span vertically or horizontally.

Basement walls are usually reinforced in both faces in both directions. This results in four layers of reinforcing. The primary tensile reinforcement is usually placed in the layer nearest the surface of the concrete in order to provide the most advantageous effective depth in the design of that reinforcing. The other three layers serve the function of shrinkage and temperature reinforcement.

Walls Designed to Span Vertically

This option can be selected in all cases where lateral load resisting elements occur at the top and bottom of the wall. A concrete floor slab will usually occur at or near the top of the wall. The wall and this slab will interact as an assembly with the gravity loads of the slab being supported by the wall and the lateral

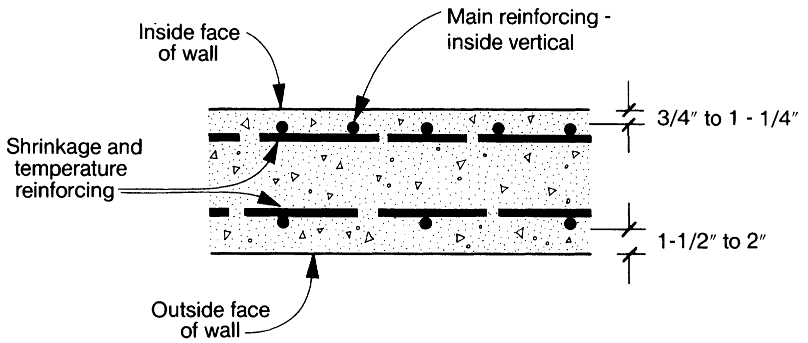


FIGURE 11-3. Typical wall reinforcing for vertical design.

pressure of the wall being supported by the slab. At the bottom of the wall, lateral support must be provided by either a continuous wall footing or a basement floor slab.

The primary tensile reinforcing of a wall designed to span verically will usually comprise the layer of reinforcement nearest the inside face of the wall. This reinforcing should extend for the full height of the wall without being spliced.

The arrangement of reinforcing that would normally be expected for a wall designed to span vertically is shown in Figure 11-3.

Walls Designed to Span Horizontally

When the architectural layout is such that main building columns are located within or adjacent to the exterior wall, the basement wall can be designed as a continuous element spanning horizontally between them.

In this case the building columns must be designed to span vertically between other elements of the building and to transfer all the earth pressure to them. The amount of earth pressure transferred into these elements by the column will, of course, be considerably more than those transferred by the action of one linear foot of wall designed to span vertically. This can result in a concentration of stress at the transfer points which must be carefully computed and evaluated by the designer.

When walls are designed to span horizontally, they are in fact continuous spans. There will be a positive moment near the center of each span and a negative moment at each support. Both layers of horizontal bars, therefore, will carry tensile stress. These bars cannot be fabricated in one continuous piece for the full length of the building, therefore, they must be spliced. Splices must be located at points where the tension in the bar is the least. Therefore, splices in the bars on the inside face of the wall will be placed at the centerline of the column, and splices in the bars on the outside face will be placed at midspan between the columns. When walls are designed horizontally the two layers of vertical bars act as shrinkage and temperature reinforcing.

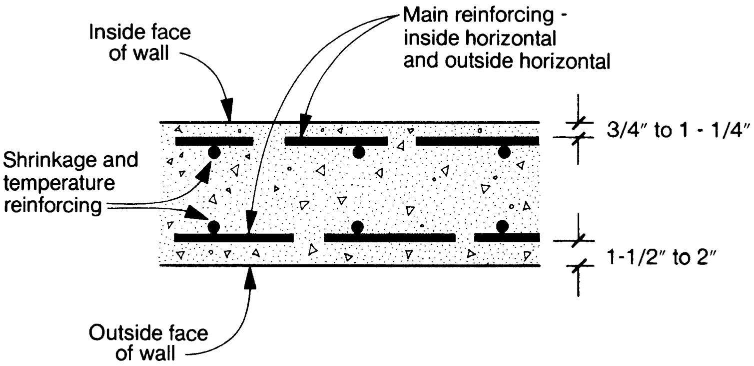


FIGURE 11-4. Typical wall reinforcing for horizontal design.

The arrangement of reinforcing that would normally be expected for a wall designed to span horizontally is shown in Figure 11-4.

General Details of Reinforcement

As previously noted, reinforcement is customarily placed horizontally and vertically in both faces of the wall. One of these layers of reinforcement is for the purpose of providing primary tensile strength to the wall in the direction of bending. The size and spacing of this reinforcing must be designed to satisfy the requirements of bending. The other three layers provide are for the purpose of providing crack control as caused by drying shrinkage or change in temperature. The size and spacing of this reinforcing need only satisfy minimum steel requirements as set forth by the ACI 318-83 Building Code. Minimum reinforcing, in accordance with that code, is given in Table 11-1.

Figures 11-5 and 11-6 show the reinforcing of a moderate height wall. Note that in each detail the outside vertical bars are bent so as to extend into the first floor slab. This is an excellent way of tying two concrete elements together. Figure 11-5 is of particular interest because of the extensive excavation. The slope of the ground surface, in all probability, closely matches the angle of

TABLE 11-1. Minimum Reinforcing for Basement and Retaining Walls

Wall Thickness (in.)	Horizontal in Each Layer	Vertical in Each Layer
8	#3 @ 12 or #4 @ 18	#3 @ 18
10	#4 @ 18	#3 @ 16
12	#4 @ 16	#3 @ 12 or #4 @ 18
14	#4 @ 14 or #5 @ 18	#3 @ 12 or #4 @ 18
16	#4 @ 12 or #5 @ 18	#4 @ 18

Note: The size and spacing of reinforcing is valid only for ASTM-A615, Grade 60 reinforcement. Table conforms to Section 14.3 of ACI 1984.

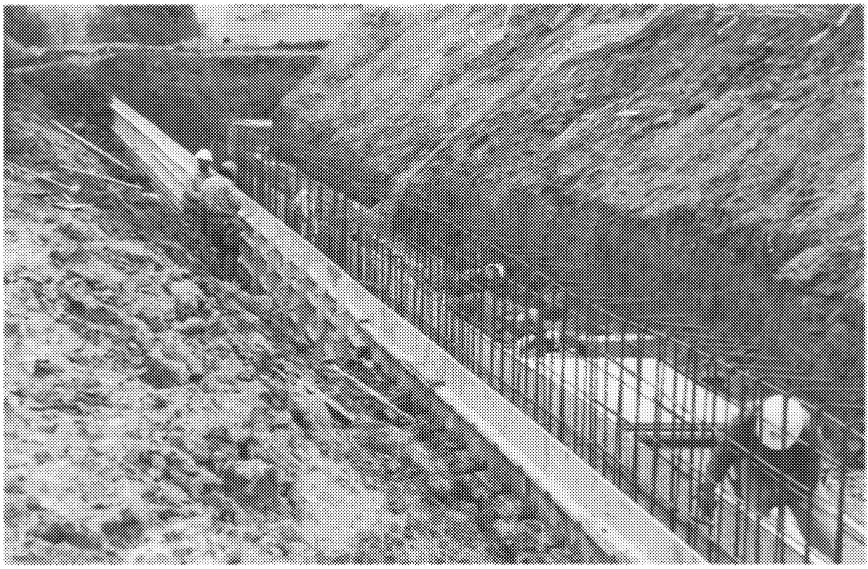


FIGURE 11-5. Reinforcement in progress for a foundation wall. Note bars bent over to extend into slab.

rupture, as illustrated in Figure 9-5. Figure 11-6 is of interest because of the large diameter sleeve installed in the formwork. This will provide an opening through the wall for the passage of various mechanical services either to enter or to exit the building.

Evaluation of Design Options

Vertical design is a more efficient and cost effective way in which to transfer earth pressure into the building structure. There are several reasons for this.

The distance from basement floor to first floor is usually less than the distance between the more widely spaced building columns. Design moments, due solely to span, will be less for walls spanning vertically.

Walls are customarily divided into one foot wide strips for purposes of design. The loads on walls designed vertically and walls designed horizontally is computed as follows, assuming that the earth pressure is without surcharge or ground water:

1. The vertically designed wall is first divided into one foot wide strips along the length of the wall. Each strip will carry a triangular load, which will vary from a maximum at the base to a minimum at the top. The intensity of this load will remain constant for each of the strips.
2. The horizontally designed wall is first divided into one foot high strips along the height of the wall. Each strip will carry a uniformly distributed

FIGURE 11-6. Sleeve built into wall to provide for mechanical access.

load. The intensity of this load will vary from strip to strip, being greater in the lower strips and smaller in the upper strips. It would be impractical to design each strip individually. Therefore, there must be a certain amount of grouping, resulting in overdesign of much of the wall.

Design moments in walls spanning vertically are a function of simple span analysis. Only one critical bending moment occurs within the height of the wall, and tensile reinforcing is required only on the inside face of the wall. Design moments in walls spanning horizontally are a function of continuous span analysis. This results in critical bending moments at two different locations. Tensile reinforcing, therefore, is required on both faces of the wall.

The shrinkage and temperature reinforcing given in Table 11-1 is not intended to act as tensile reinforcing where large bending moments occur. In areas of critical bending, therefore, additional steel must be used to satisfy the requirements of bending. Under normal conditions, it can be expected that walls designed vertically will require less additional steel.

Given his choice, the engineer will invariably elect to design the basement wall to span vertically. There are times, however, when this choice is not available to him. For a wall to span vertically, there must be lateral support at the top and bottom of the wall. When either of these supports is missing, vertical design cannot be used. Supports may be missing for the following reasons:

1. Architectural design may require the omission of certain first floor construction due to stairways, elevators or other open areas adjacent to the exterior wall.

2. Lateral support at the bottom of the wall must be provided by a continuous wall footing or by a basement floor slab. When these elements are missing, or when their use is not feasible, the wall cannot be designed to span vertically.

When vertical design is feasible, the selection of which supporting element to use at the bottom of the wall must be carefully considered. The choice between a continuous wall footing and the basement floor slab depends on an evaluation of factors pertaining to the particular building under design. Several factors that preclude the use of the slab on ground are as follows:

1. The floor slab may be subjected to climatic changes in temperature as would be the case in an open air garage. The floor must then be built free of the wall to allow for thermal movement.
2. The floor slab may lack sufficient continuity. This condition could occur in a mechanical room where there could be any number of pits, drains and trenches.
3. The construction of the floor slab cannot be completed on schedule because of a delay in the installation of under ground utilities.
4. The sequence of construction necessitates that the wall is backfilled at some time prior to pouring the slab on ground.

It should also be noted that the basement slab, when constructed as a slab on ground, is frequently the least engineered and least quality controlled element of the entire building. Once it is decided to use the slab as the restraining element, it must then be designed and built to suit that purpose. For recommendations regarding the empirical design of slab on ground refer to Appendix D.

It is usually considered better engineering to transfer the horizontal thrust of the wall into a continuous wall footing rather than into a slab on ground. The ultimate transfer of thrust back into the earth is usually considered to be more reliable when made by a footing, due to the development of passive pressure between the inside face of the footing and the earth.

The elevation of the wall footing is normally set so that the top of footing is directly beneath the stone base upon which the slab on ground is cast. There are times when the footing must be lowered because of unsatisfactory soil bearing pressure at that elevation. The footing may be lowered several feet without adversely affecting design or cost considerations.

There are instances, however, when satisfactory soil bearing pressure can only be found at a considerable depth below the basement floor. In these instances all of the building structure, including the basement floor and the basement wall, must be supported by deep foundations. It would not be practical, then, to support the basement wall on a continuous footing. The wall must now be designed to satisfy two different conditions:

1. To transfer all gravity loads to widely spaced building elements that extend down to satisfactory bearing—The wall, in this instance, will be designed as a very deep beam. Such a wall is commonly referred to as a grade beam.

2. To transfer the earth pressure into the slab on ground by designing the wall to span vertically, or to transfer the earth pressure into the building columns by designing the wall to span horizontally—Due to the buildup of lateral forces in a wall designed to span horizontally between widely spaced supports, it is evident that the preferred way to design such a wall is vertically, spanning the reasonably short distance from basement slab to first floor slab.

With grade beam construction there is no continuous wall footing. The contractor may elect, however, to pour a thin ribbon of concrete (called a footing pad) at the bottom of the grade beam in order to have a level place upon which to construct the wall forms. This type of construction was illustrated in Figure 10-16.

A Special Word of Caution

Regardless of whether the basement wall has been designed to span vertically or horizontally, certain building elements must be designed to act integrally with the wall to provide support for the lateral earth pressure to which the wall will be subjected. These elements must be in place and must be functional before any backfill is placed against the wall.

11-3. CANTILEVER RETAINING WALLS—MODES OF FAILURE

The action of cantilever retaining walls in resisting earth pressure is very different from that of basement walls. A basement wall is an inherent part of a building and transfers the earth pressure to other parts of the building with which it is in contact. The cantilever retaining wall is a free-standing wall and must transfer the earth pressure back into the soil by means of an enlarged footing.

In order to understand the workings of a cantilever retaining wall it is important to understand its modes of failure. This type of wall can fail by overturning or by sliding, as illustrated in Figure 11-7.

Overturning Mode

A wall will fail by overturning when the overturning effect of the lateral earth pressure exceeds the capacity of the wall in resistance. This resistance is a function of the following:

1. The weight of the wall, including both the stem and the footing
2. The weight of any earth situated on the footing in back of the wall and the weight of any surcharge situated on this earth, provided that this surcharge contributes to the overturning effect
3. The shearing resistance of the earth on the vertical plane located at the back edge of the footing—This resistance, being dependent upon a certain

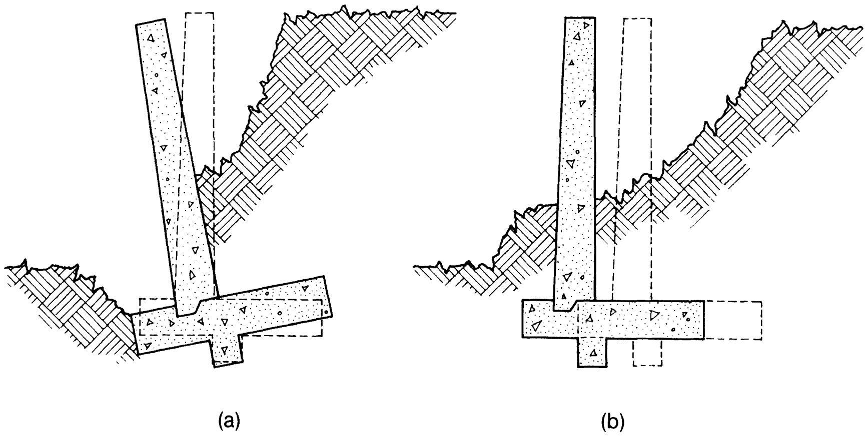


FIGURE 11-7. Failure modes of a cantilever retaining wall where (a) indicates failure due to overturning and (b) indicates failure due to sliding.

amount of deformation within the soil behind the wall, is customarily ignored in the calculations.

Sliding Mode

A wall will fail by sliding when the magnitude of the lateral earth pressure exceeds the capacity of the footing in resistance. This resistance, as developed solely by the footing, is a function of the following conditions:

1. A resistance to shear developed on the contact surface between the footing and the earth beneath. This resistance is a combination of friction and cohesion, as identified by Coulomb's equation for shearing resistance.
2. Passive earth pressure acting on the front edge of the footing and on the front edge of the shear key, if one is used.
3. In heavily loaded walls a shear key projecting from the footing into the soil beneath may be required to add to the resistance of the wall against sliding. The key should preferably be located near the toe of the footing due to the greater intensity of vertical pressure at that point. A key may also be used to provide anchorage of he main tensile reinforcing from the stem, as illustrated in subsequent Figures 11-9 and 11-15.

11-4. CANTILEVER RETAINING WALLS—DIFFERENT TYPES

Cantilever walls are typed according to where the stem is positioned with respect to the width of the footing. There are three types, as illustrated in Figure 11-8.

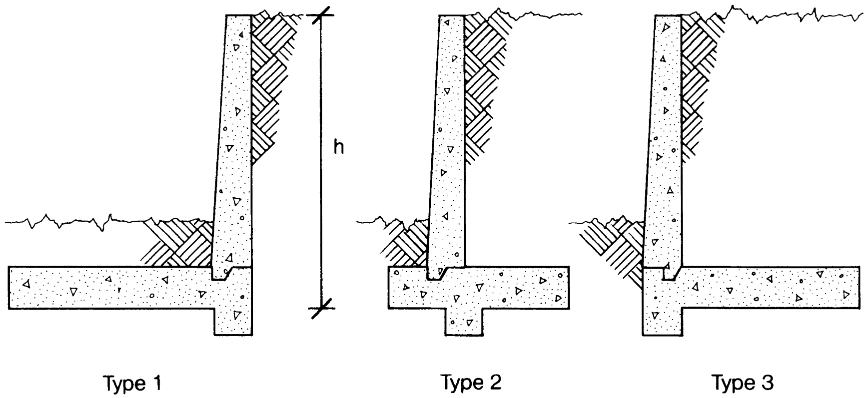


FIGURE 11-8. Three types of cantilever retaining wall. For preliminary design the width of footing may be taken as 0.70 h for Type 1, 0.50 h for Type 2 and 0.60 h for Type 3.

The width B of the footing, as given in terms of wall height, is approximately correct for walls supporting coarse grained level earth without surcharge and without ground water. These widths may increase considerably, depending upon the type of backfill and any change in loading parameters. They should therefore be used only as a guide to the initial geometry with which to start the calculations.

Type 1. Walls in this category are the least efficient of all three types of cantilever retaining walls in terms of engineering design. The earth that bears on the heel of the footings in type 2 and 3 walls contributes greatly to the resistance of the wall to overturning. The type 1 wall loses the benefit of all of this resistance and, therefore, is a very inefficient wall. This wall has the advantage, however, of requiring the least amount of excavation of all types. When excavation is costly or time consuming, as in the case of rock excavation, this type of wall may be cost effective.

Type 2. Walls in this category are the most frequently used of all three types. This is because experience has shown time and time again that they offer the best balance between engineering efficiency and cost effectiveness.

Type 3. Walls in this category are very efficient structurally because of the extension of the footing in back of the wall. This extension enables them to utilize the maximum amount of earth to resist overturning. These walls have the disadvantage, however, of requiring significantly more excavation and backfill.

General Proportions

Cantilever retaining walls must be proportioned and engineered to develop adequate resistance against overturning and lateral movement. Although there are rules of thumb relative to proportioning, the adequacy of the wall can only be determined by computations. Rules of thumb, therefore, are guidelines that give the designer a place to start. For a wall with level earth without surcharge and without ground water, the following rules of thumb should give reasonably close proportions:

1. The top of the wall should have a thickness of approximately 1″ for every foot of height, with a minimum of 8″. This rule is for the purpose of providing sufficient access between the forms for proper concreting.
2. The front face of the wall should be battered approximately ¼″ per foot. This will establish a preliminary thickness of wall at the top of the footing which should then be rounded off to some reasonable number, such as 12″, 15″, 18″, 21″, etc.
3. The width of the footings may be assumed to be approximately equal to those given in Figure 11-8, but than rounded off in 6″ increments. When the given dimension results in insufficient resistance to rotation, it must be increased, again using 6″ increments. When increasing the width of footing in a type 2 wall, proportion approximately one-third of the increase to the toe and two-thirds of the increase to the heel. Remember, the heel of the footing is on the high grade side, and the toe is on the low grade side, as indicated in Figure 10-9.
4. The thickness of the footing can be approximated as ⅛th of the overall height of the wall, rounded off to some reasonable whole number, such as 12″, 15″, 18″, 21″, etc.
5. The top of the footing should be placed no less than 12″ below finished grade.
6. The bottom of the footing must be placed at least 12″ below the lowest recorded frost depth for that locality.

11-5. CANTILEVER RETAINING WALLS—TYPICAL REINFORCING

The typical reinforcing of a cantilever retaining wall is shown in Figure 11-9. Note the the main tensile reinforcing in the stem is placed vertically along the back (or inside) face. This is because the bending moment produced by the cantilever action of the stem induces tension on that face of the wall. Similar bending also occurs in the footing. Transverse tensile reinforcing is required at the top of the heel and at the bottom of the toe. This reinforcing is customarily extended for the full width of the footing.

Note that a shear key has been provided in the soil beneath the footing. This key will provide additional passive resistance against slide. This key also provides additional length in which to develop the reinforcing. When there is no key then the bars must be hooked into the toe of the footing to provide the necessary development length.

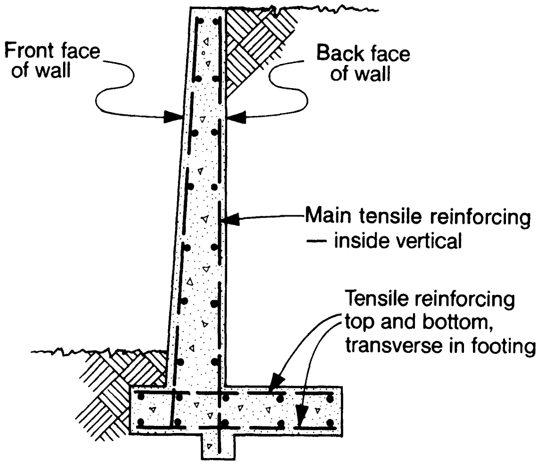


FIGURE 11-9. Typical reinforcing for a cantilever retaining wall. Shear key omitted for clarity.

Only tensile reinforcing, as required for the resistance to bending has been specified on the figure. The other reinforcing serves the purpose of shrinkage and temperature control. Minimum reinforcing, as required for any thickness of wall and footing can be taken from Table 11-1.

Figure 11-10 indicates the general reinforcing of a cantilever retaining wall under construction. Note that the vertical bars on the high grade side are substantially larger than those in the other three layers. These are the working bars and are expected to be larger and more closely spaced. Normally these bars would be place in the layer nearest the face of the concrete as indicated in Figure 11-3. That placement would develop the greater resisting moment. Why that placement was not followed in this detail is a matter of conjecture.

11-6. COUNTERFORT RETAINING WALLS

In walls up to about 20 feet in height, the vertical element of the wall, called the stem, is designed as a vertical cantilever off a footing which, because of its comparatively large size, can be considered as a fixed base. In higher walls the stem is frequently designed to span horizontally between vertical stiffening elements called counterforts. This latter type of wall is illustrated in Figure 11-11.

11-7. EARTH PRESSURE TRANSFER—CONCRETE TO CONCRETE

General

The lateral pressure exerted by the earth on a basement wall must be transferred into other elements of the building structure. This pressure is then transferred,

FIGURE 11-10. Reinforcing for a cantilever retaining wall. Note the heavy outside vertical bars.

through a variety of paths, back into the earth. It is interesting to note that the earth, which causes the problem in the first place, must also provide the ultimate solution!

A similar condition exists in the case of a cantilever retaining wall except that the stem must transfer all of the lateral pressure into the footing and from the footing into the ground.

Each path of transfer consists of individual structural elements which act together as an assembly. Because of the usual construction techniques, these elements are cast at different times. A construction joint, therefore, automatically separates these elements at their point of contact. This joint is usually called a cold joint. Earth pressures being transferred from one element to another must transfer across a cold joint.

Cold joints occur at the following locations:

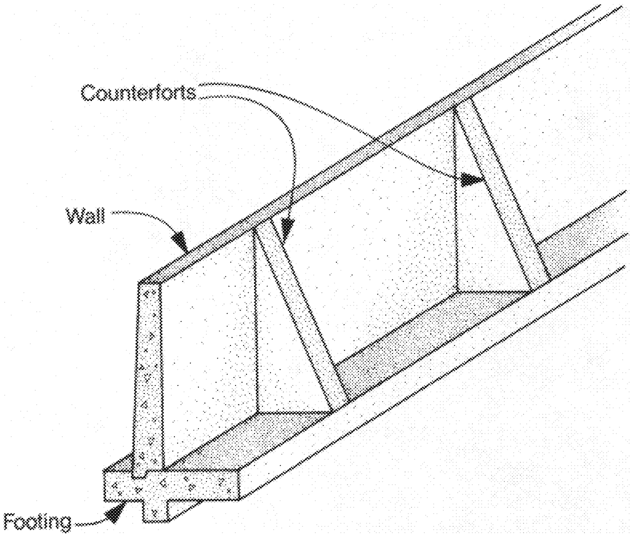


FIGURE 11-11. Typical detail of a counterfort retaining wall.

Type 1. Between basement walls and the first floor slab.
Type 2. Between basement walls and the footing, and between cantilever retaining walls and the footing.
Type 3. Between basement walls and the basement slab.

Types 1 and 2

Transfer of earth pressures across these types of joint can be analyzed by either of the following procedures, of which both are the discussed in Appendix B:

1. A method called shear-friction where transfer is made solely through the resistance of frictional forces acting on the horizontal surfaces of the cold joint. This method is discussed in Appendix A.
2. Shear key design in which transfer is made by a combination of bearing, shear and flexure. This method is analysed in Appendix B.

For the reasons discussed in the appendices, it is recommended that the shear key method of design be used for earth pressure transfer. It is further recommended that the width and depth of the shear key be proportioned so that the capacity of the key in transfer will be determined solely by pure shear and not by flexure or bearing.

The shear key method of earth pressure transfer across joint types 1 and 2 is illustrated in Figure 11-12. In these details, the planes upon which the cold joints occur are identified by heavy, solid lines. The planes through which the earth pressure is transferred by pure shear from one element to the other are indicated with dotted lines.

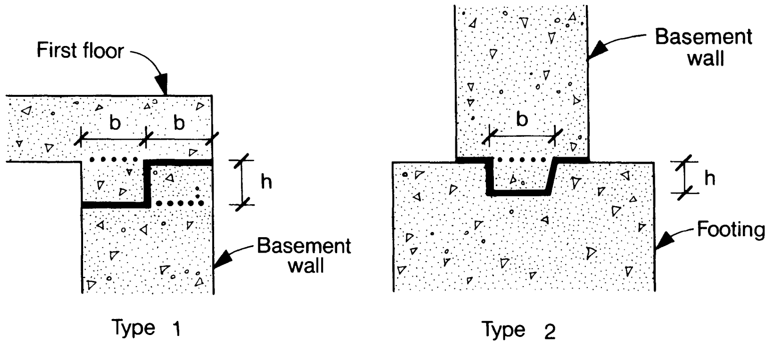


FIGURE 11-12. Cold joint details—Types 1 and 2.

The Type 1 joint is a typical joint between a basement wall and the floor above. Since there are two planes of shear, each must be at least (b) inches in width.

The Type 2 joint is a typical joint between a footing and a basement wall or the stem of a cantilever retaining wall. Note that one side of the shear key is vertical while the other side is sloped. The vertical side is the working side and transfers the lateral load through direct bearing. The other side is sloped to facilitate removal of the key form after the concrete has set.

Table 11-2 gives safe values for the force that can be transferred across a concrete shear key having a width (b) and height (h) as identified in Figure 11-12. This table was originally developed in Appendix B and is repeated here for the convenience of the reader.

TABLE 11-2. Allowable Transfer Force on Concrete Shear Keys

Nominal Size Depth × Width	Transfer Force P^a #/ft of Wall
2 × 4	1800 #
3 × 6	3240 #
4 × 8	4680 #
4 × 10	6120 #
6 × 12	7560 #

[a] Transfer forces are based on Formula (B-4), using 3000 psi concrete.

Type 3

This type of joint, as shown in Figure 11-13 is applicable to those situations when it is desirable to transfer the earth pressure from a basement wall into the basement floor slab.

The integrity of this type of joint depends on the slab being poured tight

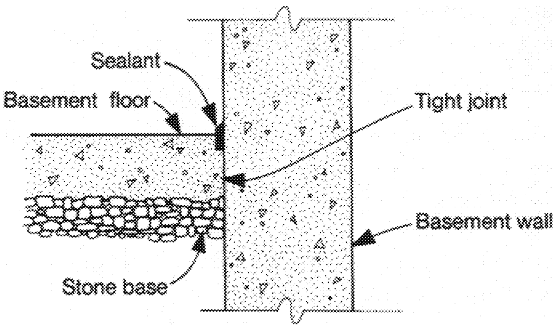


FIGURE 11-13. Cold joint detail—Type 3.

against the wall, thereby providing a surface of contact through which the load can be transferred in direct bearing. It is normally not necessary to check the resultant bearing stress on this surface, as demonstrated in the following calculation:

Consider a 5″ thick concrete slab, with 2,500 psi concrete and an allowable bearing stress $f_{brg} = 0.3\,f'_c$ from [Ref. 5]:

The resultant allowable transfer force is:

$$0.3 \times 2500 \times 5 \times 12 = 45{,}000\ \#/\mathrm{ft}$$

Obviously, this is a much higher capacity than could reasonably be required by any load transference.

11-8. EARTH PRESSURE TRANSFER—FOOTING TO GROUND

General

Earth pressure transferred from a wall to a footing must ultimately be transferred from the footing back into the ground. This latter transfer is made through a combination of two separate interactions between the earth and the footing, as follows:

1. A frictional resistance developed on the surface of contact between the bottom of the footing and the earth beneath. This resistance is commonly called shear.
2. A resistance developed on the surface of contact between the vertical side of the footing and the earth against which it bears. This resistance is called passive earth pressure and may generally be defined as the resistance

developed by a soil in response to load. The formulas involving computation of passive pressure are introduced later in this article.

These combined effects are illustrated in Figure 11-14, in which:

W = the total weight supported by the footing, including the weight of the footing

P_a = the proportionate share of earth pressure that must be transferred into the earth by the footing

F = the frictional resistance developed between the footing and the earth, as given by Formula (11-2)

p_p = the unit passive pressure developed by the earth, as given by Formula (11-3)

P_p = the total passive force developed by the earth, as given by Formula (11-4)

Note: Pressure is computed in pounds per square foot, but force is expressed in pounds per linear foot of wall. This is the customary way in which walls are analyzed and designed.

Adequate transfer of the lateral forces is essential to the performance of any earth retaining structure. It is particularly important that lateral movement of the footing is prevented. Excessive movement will cause failure. Lesser movement will cause misalignment of the structure and may induce dangerous eccentricities into the supporting elements. In order to correctly design the transfer, the classification of the soil and certain of its properties must be accurately known. These properties must be determined by laboratory analysis of undisturbed soil samples obtained from the immediate vicinity of the wall footing. The properties required are the unit weight, angle of internal friction, and cohesion.

Transfer is made through the combination of the separate resistances of shear and passive pressure. The capability of the soil to develop these resistances depends on the general classification of soil upon which the footing bears. Soil may be grouped into three major classifications:

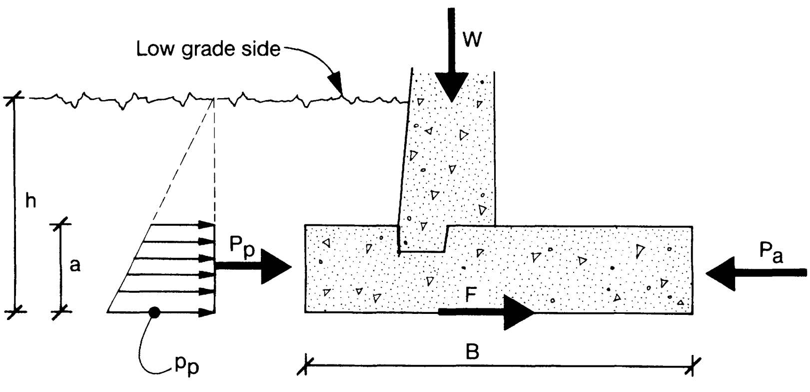


FIGURE 11-14. Earth pressure transfer from footing to ground.

1. Granular soils consisting of broken stone, gravel, sand, or nonplastic silt, found separately or in combination
2. Cohesive soils consisting of plastic silt or clay, found separately or in combination
3. Mixed grained soils consisting of a combination of coarse grained soils and cohesive constituents in various proportions

Resistance Developed by Shear

Granular soils develop resistance to shear through the physical interlocking of the various soil grains. This phenomenon is commonly called friction. Friction is also developed on the contact surface between the footing and the soil because of the inherent roughness of the soil grains. The amount of frictional resistance that can be developed between any two surfaces is a function of the degree of interlocking of the particles and the force that holds the surfaces in contact.

Cohesive soils develop their resistance to shear through a chemical bonding between particles. Cohesive soils are sticky, and the soil acts somewhat like fly paper when the footing attempts to slide over it. The magnitude of the resistance developed by a cohesive soil is independent of any contacting force and in this respect it differs greatly from that of friction.

Mixed grained soils develop resistance to shear through the combined action of friction and cohesion. The numerical value of this resistance may be computed by use of Coulomb's equation. This well known equation was first introduced in Article 4-2 and is repeated here for the convenience of the reader:

$$s = c + p \tan \phi \tag{11-1}$$

Formula (11-1) can be modified so as to be directly applicable to computing the resistance to shear of a wall footing, under the conditions illustrated in Figure 11-14:

$$F = cB + W \tan \phi \tag{11-2}$$

In which:

F = the frictional resistance
c = soil cohesion, customarily taken as one-half of the unconfined compression strength of the soil
B = the width of the footing
W = the total weight acting normal to the surface of contact
ϕ = the angle of internal friction

Note: Stresses are computed in pounds per square foot and forces are computed in pounds per linear foot of wall.

TABLE 11-3. Guidelines for Coefficients of Friction for Various Soils [Ref. 16]

Soil Description	Tan ϕ
Coarse grained soil without silt	0.55
Coarse grained soil with silt	0.45
Nonplastic silt	0.35

Note: The angle of internal friction and the unconfined compression strength refer to the in situ earth directly beneath the footing and not to the earth comprising the backfill.

For preliminary design the approximate values for tan ϕ as given in Table 11-3 may be used as guidelines. Final design, however, should be based on more accurate values as obtained from test borings and laboratory analysis.

In those instances when the soil beneath the footing consists predominantly of a medium clay, the surface should then be roughened to a depth of ⅛″ to ¼″ before the footing concrete is poured. When the soil is particularly stiff or hard, the surface should be deeply indented with a pick and shovel rather than just being roughened. The reason for doing this is to provide a physical, frictional interlocking between the footing and the soil.

Resistance Developed by Passive Pressure

Passive pressure, as illustrated in Figure 11-14, may generally be defined as the resistance developed by the soil in response to load. The resistance in this particular instance is developed on the vertical surface of contact between the footing and the soil. The magnitude of this resistance is computed as follows:

The average unit pressure taken at the mid-height of the footing is:

$$p_p = \frac{1}{2}\left[K_p\,\gamma\,h + K_p\,\gamma\,(h - a)\right]$$

$$p_p = K_p\,\gamma\left[h - \frac{a}{2}\right] \tag{11-3}$$

The total passive pressure is the product of the average unit pressure times the area; therefore:

$$P_p = K_p\,\gamma\left[h - \frac{a}{2}\right]a \tag{11-4}$$

In which:

P_p = pounds per linear foot of footing

K_p = the coefficient of passive pressure $= \tan^2\left[45° + \frac{\phi}{2}\right] = \frac{1}{K_a}$ (11-5)

K_a = the coefficient of active pressure, discussed in Article 9-5.

Note: The angle of internal friction used in passive pressure calculations relates specifically to the earth located beside the footing and not to the earth comprising the backfill.

It must be remembered that the resistance due to passive pressure can only be considered effective when it is certain that the footing will be cast against undisturbed earth whose side walls will stand without the use of wood forms. The side walls of clay soils should stand unless treated carelessly or left unprotected during a heavy downpour. The side walls of granular soils will not stand unless the area is stabilized by the injection of pressurized grout before excavation. Mixed grained soils will stand or fall, depending on the proportions of the constituents.

The contractor will usually construct an enclosure of wood forms, called screeds or screed rails, at the top of the excavation or at the top of the footing. Such screeds are illustrated in Figure 7-1. The purpose of these screeds is to precisely outline the excavation and to prevent spauling of the earth embankment. Screeds are usually only three to four inches in depth and generally have no adverse effect on the development of full passive pressure provided that the void left by the removal of the screeds is filled with concrete or compacted earth.

Additional passive pressure can be developed by the use of shear keys, as detailed in Figure 11-15. Shear keys are effective only when they are cast monolithically with the footing and when both are cast against undisturbed earth.

Some authorities have suggested that when a shear key is used to develop additional resistance in conjunction with frictional resistance,it may not be appropriate to use the full amount of frictional resistance. The author, however, has used these resistances in combination with no adverse effect, but in each case care was taken to make sure that the safety factor against slide was two or more.

Shear keys are very effective when cut into rock. Their effectiveness in soils depends on whether the side walls of the excavation will remain in tact during

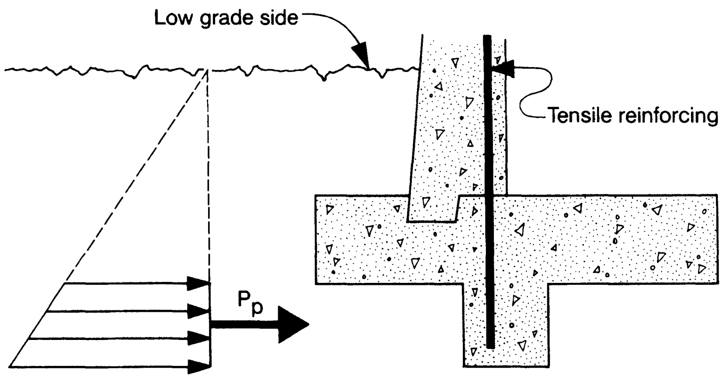


FIGURE 11-15. The development of passive pressure by shear keys extended into the soil.

the construction process. The side walls of shear keys in soils must be carefully dug by hand and then protected against cave-in until filled with footing concrete.

Safety Factor against Lateral Movement

The safety factor against a footing being pushed laterally out of position by active earth pressure may be expressed as follows:

$$\text{Safety factor} = SF = \frac{\text{total resisting force } F + P_p}{\text{total active force } P_a}$$

It should be noted that the passive pressure P_p may be the sum of the passive pressure developed against the side of the footing, as shown in Figure 11-14, and the additional passive pressure developed by a shear key, if one is installed beneath the footing, as shown in Figure 11-15.

There is general agreement between engineers and building codes that the safety factor against wall slide should never be less than 1.5. Safety factors, of course, are protection against situations having a degree of uncertainty. It is for this reason that the author recommends a minimum safety factor of 2.0. It is believed that a more conservative approach is justified because of the many intangibles encountered in the design and construction of retaining walls, and in the fickle nature of soils.

11-9. EARTH PRESSURE TRANSFER—BASEMENT SLAB TO GROUND

General Considerations

The transfer of earth pressure in a Type 3 cold joint similar to the one shown in Figure 11-13 takes the following sequence:

1. From the wall to the slab
2. From the slab to the stone base beneath
3. From the stone base to the soil

Transfer of earth pressure from the wall to the slab presents no problem. This is done through direct bearing between two concrete elements. Using an allowable bearing stress of 0.3 f'_c, the resulting resistance is far beyond any reasonable force of transfer.

Transfer from the slab to the stone base also presents no problem. The surface of the stone base, by its very nature, is very rough. A considerable amount of interlocking, therefore, will occur when the wet concrete is poured onto the stones. The concrete will also seep into the voids between the stones thereby increasing the already quasimonolithic nature of the base. Because of this dual interaction, the slab and the stone base will act essentially as a single unit to transfer the earth pressure to the ground.

Transfer of lateral pressure from the stone base into the soil beneath may present a problem. This transfer is a function of the shearing resistance developed between the two surfaces. The numerical value of this resistance may be computed by applying Coulomb's equation. Two different situations must be considered—one when the subgrade is rich in clay, the other when it is primarily granular.

As a means of evaluating these two situations, consider a 12-foot-high basement wall, supporting level earth without surcharge and without water pressure. This wall will develop an earth pressure of about 2400 #/ft, of which 1600 #/ft must be transferred at the base.

Transfer With Soils Rich in Clay

Soils rich in clay normally possess sufficient cohesion to transfer the required lateral load. A moderately stiff clay, for example, can be expected to develop an unconfined compression strength q_u of about 2 tsf, with a cohesion of about one-half that value. Using a safety factor of 3, the shearing resistance of this soil is computed as follows:

$$s = \frac{0.5 \times 2 \text{ ton/sf} \times 2000 \text{ \#/ton}}{3} = 670 \text{ psf}$$

It can be seen that the length of slab required to complete that transfer would only be about 2.4 feet per linear foot of wall.

Transfer With Granular Soil

Soils rich in sands and gravels possess little, if any cohesion. Transfer into these soils, therefore, depends almost entirely on friction. The friction being spoken of here is not that which occurs directly between the stone base and the subgrade. It is recognized that along that plane of contact there will be considerable extension and interlocking of the stones into the earth, as caused by compaction of the stone base. The plane upon which friction must be considered, therefore, is the one just below the plane of contact clear of the interlocking effect. The amount of friction that can be developed on this lower plane, according to Coulomb's equation, depends on the weight normal to the plane and the coefficient of friction along the plane. Assuming a 6″ basement slab weighing 75 psf, a 6″ stone base weighing 65 psf and an angle of internal friction of 40°, the friction developed will be:

$$F = W \tan \phi = [75 + 65] \tan 40° = 117 \text{ psf}$$

The amount of slab required for transfer would be almost 14 feet per linear foot of wall.

It can be seen that the transfer of earth pressure by friction alone may prove inadequate because the slab and stone base generally contribute insufficient weight.

This situation is indicative of a potentially serious problem that will occur whenever the soil beneath the stone base consists primarily of coarse aggregate having little or no cohesion. If this material cannot develop sufficient shear resistance then transfer cannot be made solely through friction. A different design concept for the transfer of the earth pressure must be developed.

It is evident that situations like this should be known well in advance of design commitment. This is another of the many reasons why a subsoil investigation should be scheduled well in advance of final design.

11-10. SAMPLE PROBLEMS

Example 11-1

Given: The basement wall illustrated in Figure 11-16. The wall is of vertical design, spanning from footing to first floor slab. The backfill is a medium dense, sandy gravel having the following properties:

$$\gamma = 120 \text{ pcf} \qquad \text{and} \qquad \phi = 32°$$

Required: To determine the earth pressure for which the wall must be designed, and then to design the shear keys required to transfer the lateral loads from this wall into the building structure.

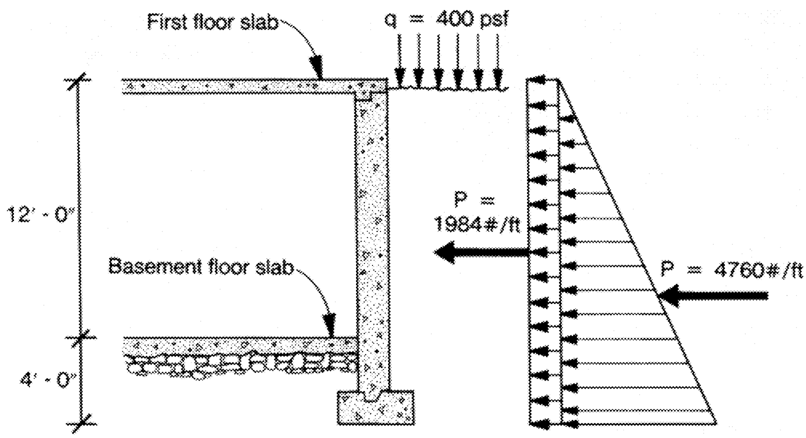


FIGURE 11-16. Example 11-1—Shear key design.

From Formula (9-5):

$$K_a = \tan^2\left[45° - \frac{32°}{2}\right] = 0.31$$

With reference to Figure 9-7:

$$\text{(a) } K_a q = 0.31 \times 400 = 124 \text{ \#/ft}$$
$$\text{(b) } K_a \gamma h = 0.31 \times 120 \times 16 = 595 \text{ \#/ft}$$

$$\text{Total force on wall} = 124 \times 16 + \frac{1}{2} \times 595 \times 16$$
$$= 1984 + 4760 = 6744 \text{ \#/ft}$$

Shear key transfer forces:

$$\text{Top of wall:} \quad P = \frac{1}{2} \times 1984 + \frac{1}{3} \times 4760 = 2579 \text{ \#/ft}$$

$$\text{Bottom of wall:} \quad P = \frac{1}{2} \times 1984 + \frac{2}{3} \times 4760 = 4165 \text{ \#/ft}$$

Note: The transfer force at the lower shear key could be reduced by the proportionate share of the footing taken in direct bearing. Normally this is an unwarranted refinement.

With reference to Figure 11-12 and Table 11-2:

Shear key at top of wall: 3 × 6 nominal; Since this is actually two shear keys side by side, the basement wall must have a minimum width equal to the sum of the width of the shear keys.

Shear key at bottom of wall: 4 × 8 nominal

Example 11-2

Given: A continuation of the basement wall from Example 11-1, in which the subgrade upon which the footing bears is a dense sandy gravel, having the following properties:

$$\gamma = 125 \text{ pcf} \qquad \text{and} \qquad \phi = 34°$$

Note that the characteristics of the subgrade are different from those of the backfill.

Required: To determine the adequacy of the footing to transfer the required lateral load in accordance through the combined action of

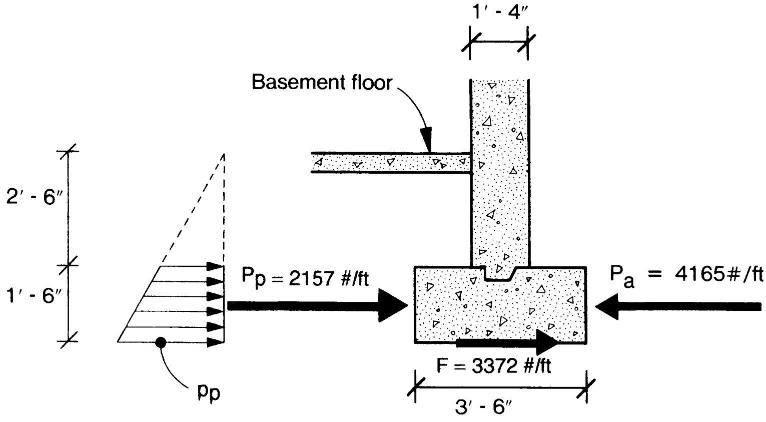


FIGURE 11-17. Example 11-2—Footing resistance to slide in sandy soil.

friction and passive pressure, as illustrated conceptually in Figure 11-14. The results of this analysis are shown in Figure 11-17.

Note: The vertical load which the footing must carry will be required in order to complete these calculations. A load of 5000 #/ft will be assumed. This load includes the weight of the first story wall, the first floor slab, the basement wall and the footing itself.

The resistance developed by shear, from Formula (11-2):

$$F = 0 + 5000 \tan 34° = 3372 \text{ \#/ft}$$

The coefficient of passive pressure is computed from Formula (11-5):

$$K_{\text{p}} = \tan^2 \left[45° + \frac{34°}{2}\right] = 3.54$$

The resistance developed by passive pressure acting on the inboard side of the footing may now be computed:

$$P_{\text{p}} = 3.54 \times 125 \times \left[4.00 - \frac{1.5}{2}\right] \times 1.5 = 2157 \text{ \#/ft}$$

The resistance due to friction combined with the resistance due to passive earth pressure provides the following safety factor against slide:

$$SF = \frac{3372 + 2157}{4165} = 1.32 < 1.50 \qquad \text{This design is unsatisfactory.}$$

Example 11-3

Required: To reexamine Example 11-2, assuming the subgrade to be a silty clay mixed with sand. The properties of this soil are:

$$\gamma = 122 \text{ pcf}, \ \phi = 18^\circ \text{ and } q_u = 1.20 \text{ tsf}$$

The resistance developed by shear:

$$F = \frac{1}{2} \times 1.20 \times 2000 \times 3.5 + 5000 \tan 18^\circ = 5824 \text{ \#/ft}$$

The coefficient of passive pressure is computed to be:

$$K_p = \tan^2 \left[45^\circ + \frac{18^\circ}{2}\right] = 1.89$$

The resistance developed by passive pressure is:

$$P_p = 1.89 \times 122 \times \left[4.00 - \frac{1.5}{2}\right] \times 1.5 = 1124 \text{ \#/ft}$$

The resistance due to friction combined with the resistance due to passive earth pressure provides the following safety factor against slide:

$$SF = \frac{5824 + 1124}{4165} = 1.67 \text{ This design is borderline satisfactory.}$$

Some codes and some soils engineers accept a 1.50 safety factor against slide as an acceptable minimum, in which case the above design is satisfactory. It is the opinion of the author, for reasons previously discussed, that serious consideration should be given to maintaining a minimum safety factor of 2.0.

EXAMPLE 11-4.

Required: To reexamine Example 11-2, assuming the subgrade to be a medium stiff clay, having the following properties:

$$\gamma = 118 \text{ pcf} \qquad \text{and} \qquad q_u = 2.20 \text{ tsf}$$

The resistance developed by shear:

$$F = \frac{1}{2} \times 2.20 \times 2000 \times 3.5 = 7700 \text{ \#/ft}$$

The coefficient of passive pressure is:

$$K_p = \tan^2\left[45° + \frac{0°}{2}\right] = 1.00$$

And the resistance developed by passive pressure:

$$P_p = 1.00 \times 118 \times \left[4.00 - \frac{1.5}{2}\right] \times 1.5 = 575 \text{ \#/ft}$$

The resistance due to friction combined with the resistance due to passive earth pressure provides the following safety factor against slide:

$$SF = \frac{7700 + 575}{4165} = 1.98 \text{ This design is considered satisfactory.}$$

Example 11-5

Given: The cantilever retaining wall having the proportions shown in Figure 11-18. These proportions were taken from the guidelines listed in Article 11-4 but were arbitrarily increased because of the addition of surcharge. The lateral and surcharge loads for which this wall must be designed are given in the same figure. These loads were originally computed in Example 9-2. In that example the density of the backfill was 110 pcf.

Required: To determine the stability against overturning of this wall.

The stability of a cantilever retaining wall is determined as follows:

1. Compute the overturning moment at point (*a*). This is the moment caused by the lateral earth pressure acting on the wall and footing.
2. Compute the righting moment at point (*a*). This moment will be due solely to the gravity loads of surcharge, earth, wall and footing.
3. Take the summation of moments about point (*a*) and determine the location at which the resultant force (*R*) intersects the base.
4. If the resultant force intersects the base within the middle third, the wall is stable; otherwise, it is not.

The overturning moment computed as force times distance, is as follows:

$$OM = 1736 \times \frac{1}{2} \times 14 + 3339 \times \frac{1}{3} \times 14 = 27734 \text{ ft\#/ft}$$

TABLE 11-4. Example 12-5—Computation of Gravity Loads and Righting Moment per Linear Foot of Wall

Item	Area	Weight	Arm	Moment	Source
1	5.0	2000	6.00	12000	surcharge
2	60.0	6600	6.00	39600	backfill
3	15.0	2250	2.87	6457	wall
4	1.5	225	2.17	488	wall
5	17.0	2550	4.25	10837	footing
		14625 #		69892 ft-#	

The gravity loads of surcharge, earth, wall, and footing, and the righting moment due to each is computed in tabular form, as shown in Table 11-4.

The vector summation of all moments about point (a) is:

$$M_a = \text{overturning moment} - \text{righting moment} + \text{weight times } (x) = 0$$
$$27734 - 69382 + 13625\ (x) = 0$$

From which (x) = 3.06 ft

Applying the middle third rule:

$$\frac{1}{3} \times 8.5 < (x) < \frac{2}{3} \times 8.5$$

Since (x) falls within the middle third of the footing width, the wall is stable against overturning.

Example 11-6

Required: To continue Example 11-5 to determine the soil pressure induced into the soil beneath the footing.

The general formula by which stresses induced by the combined action of axial and bending stress is:

$$f = \frac{P}{A} \pm \frac{Mc}{\mathrm{I}}$$

And for this example:

$$f = \frac{13625}{8.5} \pm \frac{13625\ [4.25 - 3.06]}{\dfrac{8.5^2}{6}} = 1603 \pm 1346$$

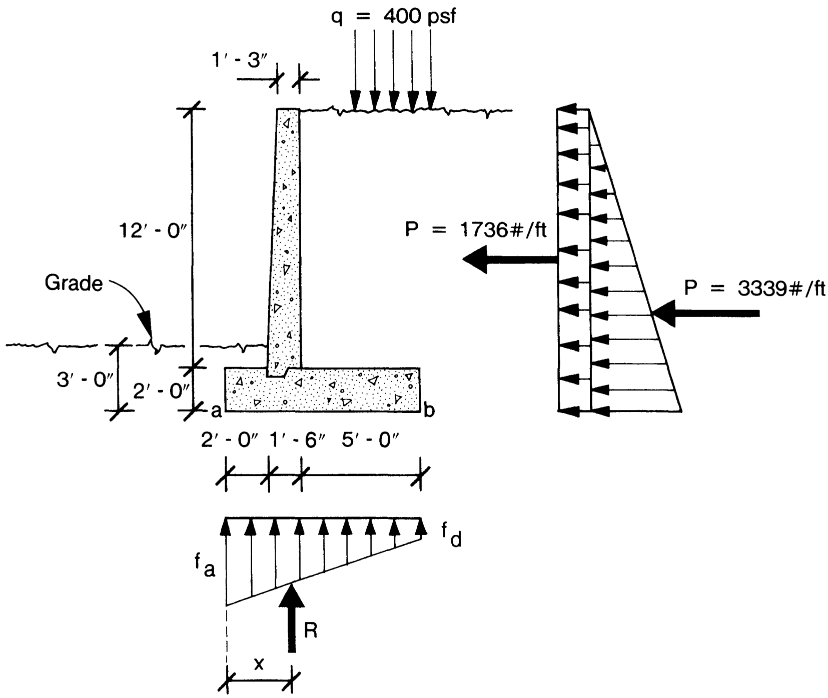


FIGURE 11-18. Example 11-5—Overturning moment and stability of a cantilever retaining wall.

Therefore: $f_a = 1603 + 1346 = 2949$ psf

and: $f_d = 1603 - 1346 = 257$ psf

This pressure gradient is plotted in Figure 11-18.

Example 11-7

Given: The subgrade into which the footing of Example 11-5 will be cast is a nonplastic, silty sand, having the following properties:

$$\gamma = 115 \text{ pcf} \qquad \text{and} \qquad \phi = 28°$$

Required: To compute the resistance against slide for this wall in accordance with the information given in Figure 11-14.

The resistance developed by shear:

$$F = 0 + 13625 \times \tan 28° = 7244 \text{ \#/ft}$$

The coefficient of passive pressure is:

$$K_p = \tan^2\left[45° + \frac{28°}{2}\right] = 2.77$$

The resistance developed by passive pressure is:

$$P_p = 2.77 \times 115 \times \left[3.0 - \frac{2.0}{2}\right] \times 2.0 = 1274 \text{ \#/ft}$$

The resistance due to friction combined with the resistance due to passive earth pressure provides the following safety factor against slide:

$$SF = \frac{7244 + 1274}{1736 + 3339} = 1.68$$

This is a borderline safety factor, and a redesign should seriously be considered.

Example 11-8

Given: A continuation of the wall in Example 11-5. In order to improve the safety factor against slide an 18-inch-deep key has been added, as detailed in Figure 11-19.

Required: To determine the effect of the shear key on resistance to slide.

The passive pressure, computed from Formula (11-4) is:

$$P_p = 2.77 \times 115 \times 3.5\left[4.5 - \frac{3.5}{2}\right] = 3066 \text{ \#/ft}$$

Combining this passive pressure with the 7244 #/ft due to friction, as determined in Example 11-7, the safety factory against slide, is:

$$SF = \frac{3066 + 7244}{5075} = 2.03$$

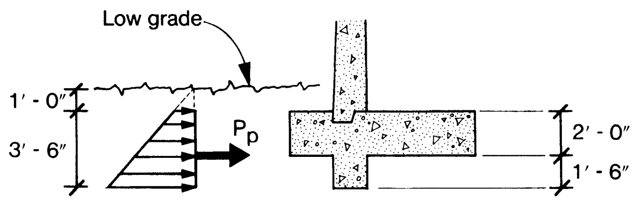


FIGURE 11-19. Example 11-8—Passive pressure developed by footing and a shear key extending into the soil.

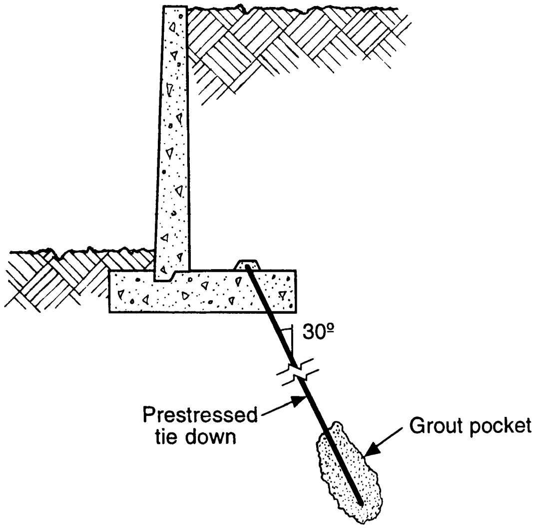


FIGURE 11-20. Example 11-9—Cantilever retaining wall stabilized by a post-tensioned tiedown system.

The addition of the shear key increased the safety factor substantially, and this design should be considered satisfactory.

Example 11-9

Given: A continuation of the wall in Example 11-5. A steel tiedown has been added, as shown in Figure 11-20. The tiedown is a seven wire strand, low relaxation prestressed tendon having a nominal diameter of ½ inch. Other properties are as follows:

$$A_{ps} = 0.144 \text{ si} \qquad f_{pu} = 250 \text{ ksi} \qquad f_{py} = 225 \text{ ksi}$$

Required: To determine the effect that the tiedown will have on the resistance to slide.

The analysis of a post-tensioned tendon requires a examination of the stresses induced during the following stages:

1. The jacking stage: This is when the force applied to the tendon is maximum. The maximum allowable force at this stage is the lesser of the two following criteria:

$$0.94 \times f_{py} \times A_{ps} = 0.94 \times 225 \times 0.144 = 30.4 \text{ kips}$$
$$0.85 \times f_{pu} \times A_{ps} = 0.85 \times 250 \times 0.144 = 30.6 \text{ kips}$$

2. The transfer stage: As the load is transferred from the jack to the concrete by securing the anchorage devices, there is a loss in tendon tension of approximately 10%. Assuming the tendon can be jacked to the maximum permissible force as found in item 1, then the prestress remaining after transfer would be:

$$30.4 \times 0.90 = 27.4 \text{ kips}$$

But the maximum force permitted in the tendon by code at this stage is:

$$0.70 \times f_{pu} \times A_{ps} = 0.70 \times 250 \times 0.144 = 25.2 \text{ kips}$$

Since the permissible force in the cable at this stage is less than what would result from a maximum jacking force, the original jacking force must be reduced.

$$\text{Maximum jacking force} = \frac{25.2}{27.4} \times 30.4 = 28.0 \text{ kips}$$

This is the force for which the grout pocket must ultimately be designed.

3. The service load stage: It is the nature of a tendon to exhibit a loss of strength during a period of years at the beginning of the service load stage. This loss of strength, which averages about 15%, is similar to the phenomenon of creep in concrete.

   The long-term prestress available, assuming a 15% loss during the service load stage will be:

$$25.2 \times 0.85 = 21.4 \text{ kips}$$

This is the force that can be used to develop additional resistance to footing slide.

Assuming the cable to be at a 30° angle measured from the vertical and assuming that the cables are placed on five foot centers along the wall, the components of the 21.4 kip force are as follows:

$$H = 21{,}400 \times \sin 30° \times \frac{1}{5} = 2140 \text{ \#/ft}$$

$$V = 21{,}400 \times \cos 30° \times \frac{1}{5} = 3706 \text{ \#/ft}$$

The cable helps develop lateral resistance in two separate actions:

1. The horizontal component acts directly to increase resistance.
2. The vertical component acts additively with the other vertical loads to increase frictional resistance.

The additional resistance to slide on a per foot basis is equal to:

$$2140 + 3706 \tan 28° = 4110 \text{ \#/ft}$$

If this effect were applied to Example 11-7, the total resistance to slide would be:

$$8518 + 4110 = 12{,}628 \text{ \#/ft}$$

With a corresponding safety factor against slide of:

$$\text{SF} = \frac{12628}{5075} = 2.48$$

Note, also, that the vertical component of this cable will increase the contact pressures f_a and f_d, as computed in Example 11-6.

Example 11-10

Given: The tendon previously analysed in Example 11-9, the maximum jacking force of which was determined to be 28.0 kips. The tendon will be anchored in a grout pocket, installed in a 4-inch bore hole drilled into sound rock. Grout will have an ultimate compression strength of 4000 psi.

Required: To determine the required length of the grout pocket, based on the stresses given in Article 10-12. The average value of these stresses will be used for the purpose of this demonstration. Stresses for actual construction should be determined by test.

1. Compute the pocket length required to transfer the force from the tendon to the grout. The pocket length is a function of the ultimate bond strength applied to the contact surface between the grout and a one-half inch diameter tendon. Use a safety factor of 2.

$$\frac{\text{Ultimate bond strength}}{\text{safety factor}} = \frac{\text{Force}}{\text{bar perimeter} \times \text{pocket length}}$$

$$\frac{300 + 800}{2} = \frac{28000}{\pi\, 0.5\ \text{L}\ 12} \qquad \text{So: } L = 5.4 \text{ feet}$$

2. Compute the pocket length required to transfer the force from the grout to the rock. The pocket length is a function of the working bond strength applied to the contact surface between the grout and a 4-inch bore hole.

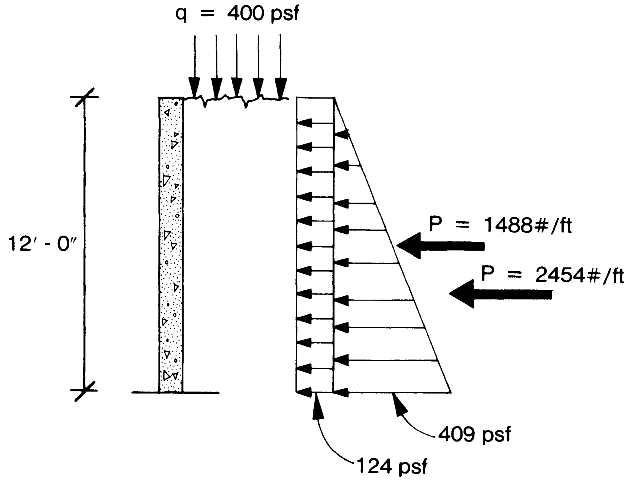


FIGURE 11-21. Example 11-11—Moment induced into stem of the cantilever retaining wall.

$$\frac{50 + 200}{2} = \frac{28000}{\pi\ 4.0\ L\ 12} \qquad \text{So: } L = 1.5 \text{ feet}$$

The grout pocket would therefore be specified as 5–6″ in length.

Example 11-11

Required: To compute the design moment for the stem of the cantilever retaining wall originally introduced in Example 11-5. Refer to Figure 11-21.

$$p_1 = 0.31 \times 400 = 124 \text{ psf}$$
$$\text{From which: } P_1 = 124 \times 12 = 1488 \text{ \#/ft}$$
$$p_2 = 0.31 \times 110 \times 12 = 409 \text{ psf}$$
$$\text{From which: } P_2 = 0.5 \times 409 \times 12 = 2454 \text{ \#/ft}$$

The maximum moment in the stem occurs at the top of footing:

$$M = 1488 \times 6 + 2454 \times 4 = 18{,}744 \text{ ft\#/ft}$$

Example 11-12

Required: To determine the moments induced into the footing of the wall examined in Example 11-5. Refer to Figure 11-22.

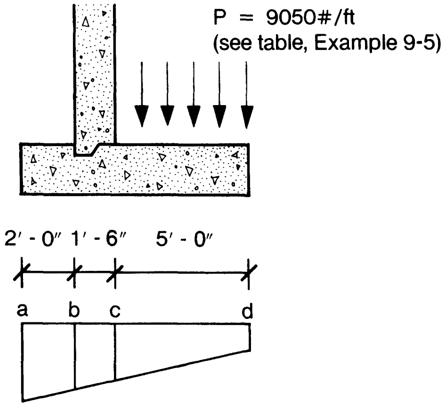


FIGURE 11-22. Example 11-12—Moment induced into footing of the cantilever retaining wall.

From Example 11-6, the pressures at points a, b, c, and d:

$$f_a = 2949 \text{ \#/ft}$$
$$f_b = [2949 - 257]\,\frac{6.5}{8.5} + 257 = 2315 \text{ \#/ft}$$
$$f_c = [2949 - 257]\,\frac{5.0}{8.5} + 257 = 1840 \text{ \#/ft}$$
$$f_d = 257 \text{ \#/ft}$$

The moment at point (*b*) equals the moment of the trapezoidal earth pressure pushing up minus the moment of the weight of the footing pushing down. This moment produces tension on the bottom face of the footing. Note that the weight of earth above the footing is ignored.

$$M_b = 2315 \times 2.0 \times 1.0 + 0.5 \times [2949 - 2315] \times 2.0 \times 1.33$$
$$- 300 \times 2.0 \times 1.0 = 4873 \text{ ft\#/ft}$$

The moment at point (c) equals the moment of the surcharge and earth above the footing plus the moment of the weight of the footing minus the moment of the trapezoidal earth pressure beneath the footing. This moment produces tension at the top face of the footing.

$$M_c = [400 + 110 \times 12 + 300] \times 5.0 \times 2.5 - 257 \times 5.0 \times 2.5$$
$$- 0.5 \times [1840 - 257] \times 5.0 \times 1.67 = 15{,}428 \text{ ft\#/ft}$$

12

Soil Compaction

12-1. GENERAL

Soil compaction is a process by which a layer of soil is mechanically densified. This process will significantly alter the physical characteristics of the soil in the following ways:

1. The thickness of the layer will be decreased. This will be accompanied by a corresponding increase in unit weight.
2. The shear strength and the load bearing capacity will be increased.
3. For a given load the settlement of the compacted soil will be substantially less than that of the uncompacted soil.

These effects are all beneficial in that the net result is to increase the capability of the soil to support load. It should be noted, however, that compaction will also decrease the permeability of a coarse grained soil.

The process of compacting a soil depends on whether the soil is coarse grained or fine grained. With coarse grained soils compaction is primarily accomplished by rearranging the seating of the granular particles. Occasionally fines are introduced throughout the mix to improve particle stability. With fine grained soils compaction is the result of consolidation. Consolidation is a term used to specifically describe the compaction of a fine grained soil by the process of squeezing air and water out of the voids. The term compaction is sometimes confused with consolidation, since the soil is densified in both processes. Compaction refers to the overall process of densification. Consolidation relates solely to the compaction of fine grained soils.

Compaction is the result of a man-made effort performed either in the laboratory or in the field. With coarse grained soils, this is a rather quick process, being measured in minutes or hours. With fine grained soils, consolidation is a relatively slow process. When performed in the laboratory, it is measured in days, but when it is the result of a natural occurrence such as glacial ice or volcanic sediment, consolidation may be measured in thousands of years.

It has been well documented that the best way to compact earth is to compact it in layers, with each layer being limited to approximately 8 to 12 inches in thickness. Fine grained soils compact less readily than granular soils. It can be expected, therefore, that soils rich in clay or plastic silt should be compacted in thinner layers than soils having granular characteristics. The optimum thickness with which to compact any soil can be predetermined by running compaction tests on several different thicknesses and comparing results.

Relatively thick masses of soil can be produced by compacting one layer on top of another in sequence. Before a new layer of earth is deposited on a freshly compacted layer, the surface of the compacted layer should be scarified so as to produce a mechanical bonding between the surfaces.

Proper compaction is not a hit-or-miss proposition. The architect, who has the ultimate responsibility for writing the specifications, should consult with the project engineer, the testing laboratory and the contractor for their advice concerning the technical and performance areas of the work before committing himself to any course of action.

12-2. BORROW FILL

Borrow fill is a term used to identify soil that must be brought from its original location to the place where it will be used. Borrow fill may be required because of any one of the following conditions:

1. To raise the existing grade to meet architectural design requirements
2. To replace soil having unsatisfactory bearing capacity
3. To use in all areas where soil is required and the on-site soil does not have the required characteristics

When borrow fill is required, the architect must make the decision as to which type of soil should be specified. In making this decision, only those soils that are proper for the intended use should be considered, but this decision must also be influenced by the availability and cost effectiveness of the different types of soil.

An examination of the soil groups listed in Table 12-3 provides insight as to the particular characteristics of each soil with regard to density, compaction, stability, bearing capacity, settlement, and expansion. Soil groups designated GW, GP, GM, and SW are the best from the standpoint of quality assurance during compaction and can be expected to produce a well densified soil mass. For all-around excellence as a borrow fill, these are the soil groups from which the architect should limit his selection. In those instances where bearing capacity is not of primary concern, soil groups SP and SM could reasonably be specified.

FIGURE 12-1. Soil being loaded at a borrow pit. [Ref. 7]

Borrow fill is obtained from natural deposits of soil whose physical properties are equal to, or similar to, those required by the architect. In some instances the soil may be used as dug from the borrow pit. In other instances, it may be necessary to screen the soil or to mix it with other soils. Figure 12-1 shows the soil being loaded at the borrow pit for transport to the site of the building.

After the material for use as borrow fill has been selected, the maximum possible density of this material should be determined by laboratory analysis. The architect, possibly working in conjunction with his engineer, will then determine the required percentage of this density to be achieved during compaction. Compaction procedures consistent with the required field density must be established, and field inspection of the work in progress must be made by the architect or his representative in order to determine procedural compliance. The testing laboratory must obtain and test samples of the in-place soil to determine compliance with the specification requirements. Inspection and testing should be performed on a continuing basis until it is evident that the work is being performed satisfactorily.

12-3. SITUATIONS WHERE SOIL COMPACTION IS REQUIRED

Compaction serves a very necessary function in almost all architectural engineering projects, in both the preparation of the site and in the actual construction of the building. The particular situations in which compaction is required are as follows:

1. When a relatively thin layer of existing soil is too loose or too soft to provide adequate bearing, but the soil below is of sufficient density to provide the required bearing. Compaction, in this instance, will densify the upper layer of soil and increase its load bearing capacity.
2. When the existing earth can provide adequate bearing, but must be built up to meet new elevations as established by architectural design requirements. In this instance borrow fill must be brought to the site and compacted in place.
3. When the backfill placed against basement walls and retaining walls must meet certain materials specifications and compaction requirements. These requirements were discussed in Article 10-13.
4. When the earth has been excavated below final grade or below the slab on ground in order to install spread footings or pile caps. All such areas must be backfilled with approved material and compacted in place.
5. When areas have been overexcavated, either in width or in depth, or when areas adjacent to footings have side walls collapse or spall off before concrete is poured. This is a particular problem in which wall footings are stepped, as in Figure 7-4. At the contractor's option, these latter areas may be filled with footing concrete, rather than being backfilled with earth and compacted.

In the related field of civil engineering and site development, compaction plays a very important role in the construction of highways, embankments, earthen dams, soil conservation, landfill, and soil reclamation.

12-4. COMPACTION OF LARGE, OPEN AREAS

The procedures used in the compaction of relatively large, open areas are different from those used in the compaction of small, confined areas.

Roads, driveways, basement slabs and exterior paving are examples of large, open areas. These areas are indicative of the situations described in items 1 and 2 of the previous article. Because these areas are relatively large and open, they lend themselves to the use of heavy earth moving and compaction machinery. For those instances where borrow fill is required, the approved soil is brought to the site and either stockpiled for future use or spread out over the required area immediately. Care must be taken to spread the soil out in layers not exceeding the optimum thickness established for the particular soil being compacted. The spreading out of soil in layers is illustrated in Figure 12-2.

Compaction of large, open areas can be done with the use of a variety of heavy machinery equipped with rollers. Some machinery is motorized and moves

FIGURE 12-2. Spreading soil before compaction. [Ref. 7]

over the area under its own power. Other machinery is not motorized and must be towed. All this machinery, however, is designed to cover large areas of soil quickly and to exert very large contact pressures.

The contacting elements consist of heavy rollers. There are two general types of rollers. Figure 12-3 illustrates a smooth drum roller. Figure 12-4 illustrates a paddlefoot roller, which is frequently called a sheepsfoot roller. Both rollers may be mechanically vibrated in order to increase their effectiveness in compaction. The paddlefoot drum roller offers the advantage of scarifying the surface of each layer, thus providing a mechanical bond between layers. A scarified subgrade is shown in Figure 12-5.

Not all soils respond equally to compaction by the same type of roller. One of the tricks to the achievement of proper compaction is to select the roller that is most effective for the particular soil being compacted. Considerable work has been done in this area by the Army Corps of Engineers. For their recommendations as to the proper matchup of soil and roller, refer to the compaction characteristics of various unified soil groups, as given in Article 12-11.

12-5. COMPACTION OF SMALL, CONFINED AREAS

Small, confined areas requiring compaction were described in items 3, 4, and 5 of Article 12-3. These areas require great care in the depositing of the soil so as

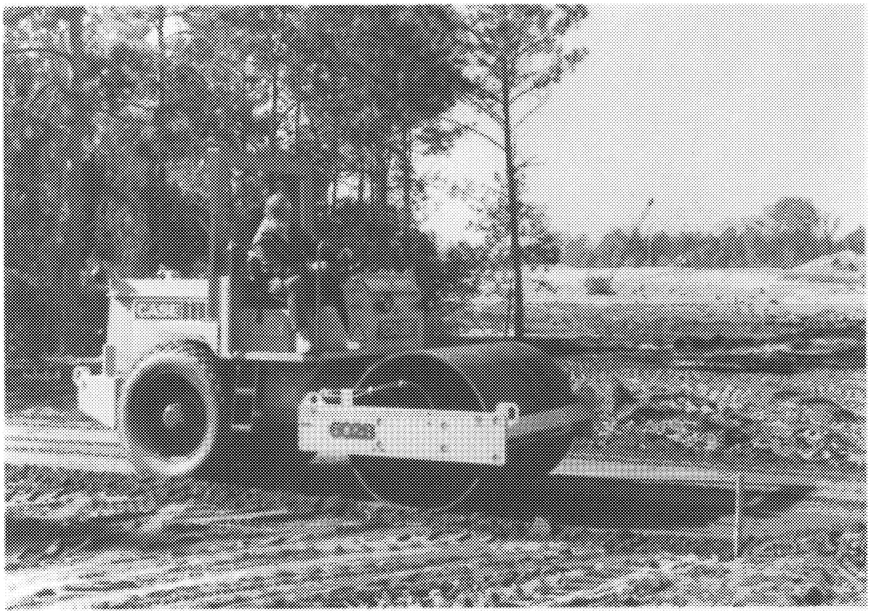


FIGURE 12-3. A smooth drum vibratory roller. [Ref. 7]

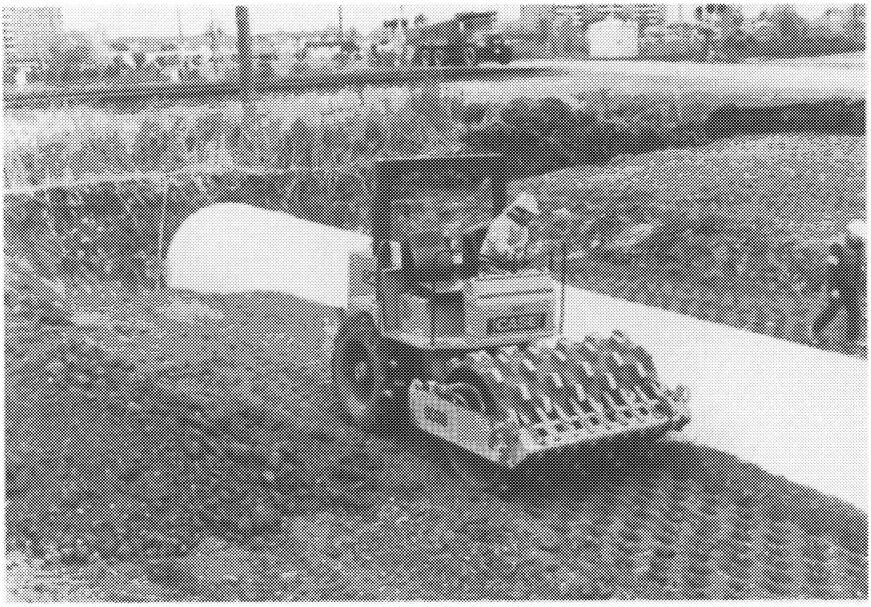


FIGURE 12-4. A paddlefoot drum vibratory roller. [Ref. 7]

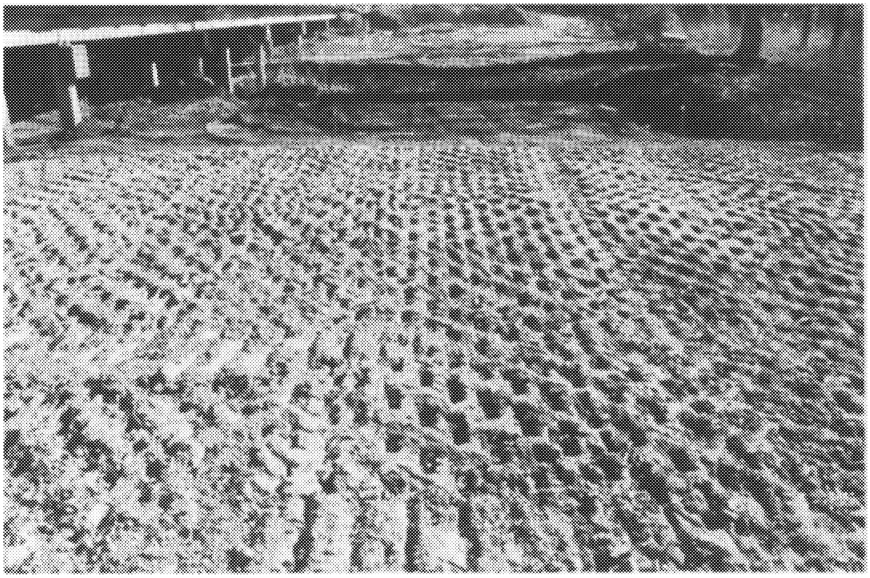


FIGURE 12-5. Subgrade scarified with a paddlefoot drum roller.

not to damage adjacent construction. This work can sometimes be done with a front end loader but at other times must be done by hand.

Compaction in these small areas follows the same basic procedures as those described for large areas. This includes the determination of optimum layer thickness, moisture control, and in situ density tests. The main difference in performing the work in small areas is that the work must be done by hand. Two basic kinds of portable compaction equipment are available for that purpose. Both kinds are illustrated in Figures 12-6 and 12-7.

12-6. COMPACTION OF COARSE GRAINED SOILS

General

The theory by which compaction works for a coarse grained soil is entirely different than that for a fine grained soil. Coarse grained soils exist by their very nature in intergranular contact, much like a bucket of marbles. The way in which these grains are arranged within the mass and the distribution of particle size throughout the mass will ultimately determine the density, the stability and the load bearing capacity of that particular soil.

The honeycombed structure shown in Figure 12-8(a) is representative of very poor intergranular seating. Such a structure is inherently unstable and can collapse suddenly when subjected to shock or vibration. The stability and load bearing

FIGURE 12-6. A hand operated vibratory rammer. [Ref. 7]

capacity of this type of soil will be improved by compaction because of the resulting rearrangement in intergranular seating. With sufficient compaction this structure will take on the characteristics of the arrangement shown in Figure 12-8(b).

The arrangement of particles shown in Figure 12-8 provides maximum intergranular contact, but there are insufficient fines to lock the larger particles in place. Compaction of this type of arrangement is ineffective since neither additional particle contact nor additional stability can be achieved. This soil is inherently stable, however, when it is laterally restrained and demonstrates good load bearing characteristics. When insufficiently restrained, however, this soil will be free to move laterally resulting in a pronounced loss in stability and load bearing characteristics. Such a soil should not be considered as a serious load bearing element.

The arrangement of particles shown in Figure 12-8(c) not only provides maximum intergranular contact but also inherent stability. This very important property

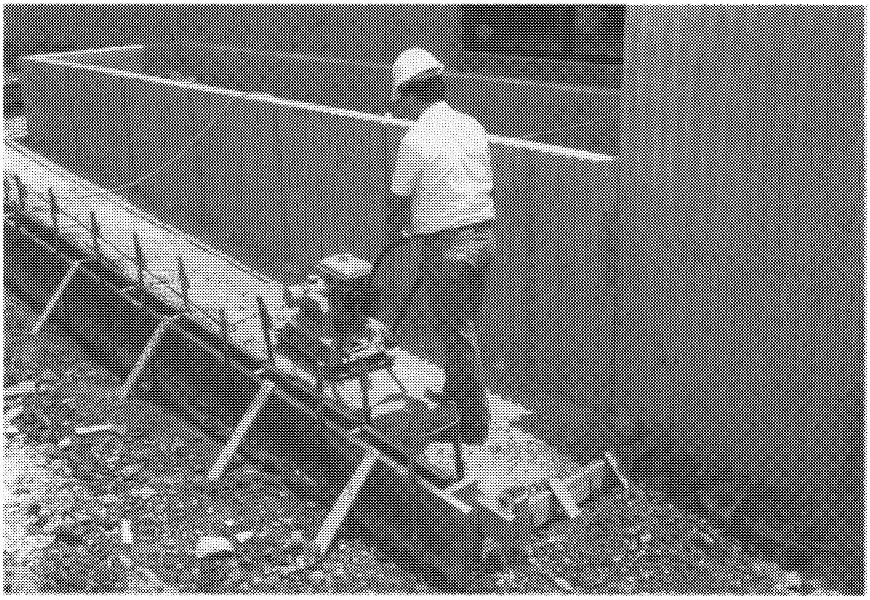
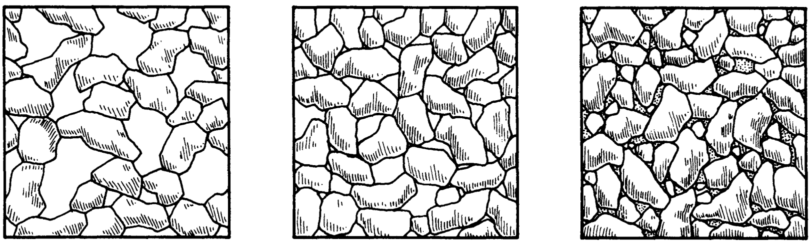


FIGURE 12-7. A self-propelled plate compactor. [Ref. 7]

of stability is due to the inclusion of fines in the spaces between the larger particles. When these particles are well graded, as identified in the particle distribution curves in Figures 1-3 and 1-4, the stability of the mass will be the best that it can be. There is one cautionary note that must be made concerning fines: too many fines are detrimental to the mix, as they may separate the larger grains, destroying the intergranular contact between them. In this instance, the larger grains are more or less floating in a sea of fines.

The intergranular seating of a coarse grained soil can be improved by the process of compaction. Particle distribution can be improved by the physical addition and mixing of fines into the soil. Both separate actions increase the



(a) (b) (c)

FIGURE 12-8. Intergranular seating and gradation of particles: (a) poorly graded, poorly seated, (b) poorly graded, but well seated, (c) well graded and well seated.

density of the soil. This increase in the soil's density can be used as a measure of the success of each of the two operations. Density is a function of the amount of voids contained within a given volume of soil. The potential for a soil to be further densified depends upon how much of a reduction can be made in the void ratio. This reduction is not without limit. Every mixture of granular material inherently has a minimum void ratio (maximum density) and for a given mixture this ratio cannot be changed. Once a soil has been compacted to its maximum density, continued efforts at compaction will only result in the crushing of the individual grains.

Compaction in Terms of Relative Density

Compaction of coarse grained soils is considered satisfactory when the relative density of the soil in place is no less than some specified percentage of its maximum possible density. Relative density, it may be remembered, is a term used to compare numerically the density of an in-place natural or compacted soil with the densities represented by the same soil in the extreme states of looseness and denseness. For the conditions of loading normally found in and around buildings, the values given in Table 12-1 are considered reasonable guidelines.

TABLE 12-1. Recommended Minimum Relative Density of a Coarse Grained Soil as a Function of Intended Use

D_r (%)	Location
85–90	Backfill against basement walls and retaining walls that support exposed earth without surcharge
90–95	Earth supporting lightly loaded building elements, such as basement floors and incidental paving
95–100	Earth supporting heavily loaded building elements, such as mechanical rooms and loading docks
95–100	Earth supporting private roads and driveways
100	Earth supporting major building loads and any other areas where settlement must be carefully controlled

Note: These values guidelines only. The values to be used on any project must be determined by the architect or engineer in charge of the work.

Determination of Relative Density

Reference is made to Figure 2-3, which indicates the variation in density as a function of void ratio.

There are two methods by which the relative density of a given soil may be determined. Both methods require the performance of a laboratory analysis.

Method 1: Void Ratio. This method was introduced in Article 2-8. Void ratios must be determined for the three conditions of the soil: (a) loosest, (b) in-place, and (c) densest. The relative density is then computed by Formula (2-8). Example 2-3 illustrates the calculations required for this method.

Method 2: Unit weight. Relative density can be expressed in terms of unit weight, with the following formula:

$$D_r = \frac{\gamma_{nat} - \gamma_{min}}{\gamma_{max} - \gamma_{min}} \times \frac{\gamma_{max}}{\gamma_{nat}} \times 100\% \qquad (12\text{-}1)$$

In which:

γ_{max} = the dry unit weight of the soil in its densest state
γ_{nat} = the dry unit weight of the soil in place
γ_{min} = the dry unit weight of the soil in its loosest state.

It is the customary practice to use oven dry samples for all of this work. There may be some question as to the need to do this due to the very small percentage of water in a sample of well drained, coarse grained soil. Soil near the water table, on the other hand, may contain appreciable water due to the phenomenon of osmosis.

Formula (12-1) is the result of transforming Formula (2-8) by substituting unit weight equivalents for void ratios as shown herein:

$$\text{Using: } \gamma = \frac{W_s + W_w}{V_s + V_v} \qquad (2\text{-}1)$$

And noting that $W_w = 0$ because dry weights are being used, then:

$$V_v = \frac{W_s}{\gamma} - V_s \qquad \text{and} \qquad e = \frac{V_v}{V_s} = \frac{W_s}{\gamma V_s} - 1$$

The above general expression for (e) can be adapted to satisfy the three different expressions found in Formula (2-8), namely e_{max}, e_{min}, and e_{nat}, by using the unit weight corresponding to each void ratio. Noting that minimum density corresponds to maximum void ratio, etc, then:

$$D_r = \frac{\dfrac{W_s}{\gamma_{min} V_s} - 1 - \dfrac{W_s}{\gamma_{nat} V_s} - 1}{\dfrac{W_s}{\gamma_{min} V_s} - 1 - \dfrac{W_s}{\gamma_{max} V_s} - 1}$$

From which: $$D_r = \frac{\dfrac{1}{\gamma_{min}} - \dfrac{1}{\gamma_{nat}}}{\dfrac{1}{\gamma_{min}} - \dfrac{1}{\gamma_{max}}} = \frac{\dfrac{\gamma_{nat} - \gamma_{min}}{\gamma_{min}\,\gamma_{nat}}}{\dfrac{\gamma_{max} - \gamma_{min}}{\gamma_{min}\,\gamma_{max}}}$$

Finally: $$D_r = \frac{\gamma_{nat} - \gamma_{min}}{\gamma_{max} - \gamma_{min}} \times \frac{\gamma_{max}}{\gamma_{nat}} \times 100\% \qquad (12\text{-}1)$$

12-7. COMPACTION OF FINE GRAINED SOILS

General

The theory by which compaction works for a fine grained soil is entirely different from that for a coarse grained soil. The reason for this is that fine grained soils possess cohesion. It should be remembered that the finer fraction of the fine grained soils exists in a colloidal state, and all colloids possess cohesion. The mineral grains of a cohesive soil are not in physical contact as they are in a coarse grained soil. Every grain is surrounded by an extremely thin blanket of water; these water molecules are electrically bonded to the grains. This blanket of water isolates the grains and prevents them from being in physical contact with adjacent grains. For a discussion of this phenomenon, refer to Chapter 13.

The degree to which a fine grained soil can be compacted is almost wholly dependent on the in situ moisture content of the soil. The moisture content that corresponds to the maximum degree of compaction is called the optimum moisture content. The approximate optimum moisture content of several soil groups is given in Table 12-2.

TABLE 12-2. Approximate Range of Optimum Moisture Content [Ref. 13]

Soil Type	Optimum Moisture Content (%)
Sand	6–10
Sand–silt	8–12
Silt	11–15
Clay	13–21

Note: The terms moisture content and water content are synonymous and are used interchangeably in industry and in this text.

Proctor Density Tests

The interaction between density and moisture content came to the attention of an engineer by the name of R.R. Proctor in the early 1930s. Proctor was involved in work regarding the densification of earth dams, and during this work he developed a procedure whereby the interaction between density and moisture content could be determined in the laboratory. This procedure is known throughout the industry as the Proctor Density Test. The American Society for Testing and Materials has adopted this procedure, with the following identification:

> ASTM Designation D-698: Standard Test Methods for Moisture-Density Relations of Soil and Soil-Aggregate Mixtures Using 5.5# Rammer and 12″ Drop

A similar procedure, ASTM Designation D-1557, has also been adopted by the American Society for Testing and Materials. This procedure, known as the Modified Proctor Density Test, gives essentially the same results but utilizes a somewhat different methodology.

The purpose of performing either of the Proctor tests is to determine the variation in soil density as a function of moisture content. This work, as performed under very exacting conditions in a laboratory, will establish the maximum density to which a given soil can be compacted for a number of different moisture contents. When the results of this test are plotted, the following data can be readily determined:

1. The maximum density to which this particular soil can be compacted
2. The range of moisture content through which any specified density can be achieved

An abbreviated description of the basic methodology of the Proctor Density Test is included herein. The reader may wish to refer to one of the ASTM standards for a more in-depth discussion.

The apparatus required for this test is precision built to meet rigid standards. It is constructed in two parts, as indicated in Figure 12-9. The lower part is called the mold, and the upper part is called the collar or extension. The purpose of having a removable collar is to provide a place to hold the loose soil before the mold is filled with compacted soil. Molds are manufactured in two different sizes based on diameter and volume. The first is a 4-inch mold having a volume of 1/30 CF, and the other is a 6 inch mold having a volume of 0.075 CF. The 4-inch-diameter mold is detailed in Figure 12-9.

There are four methods by which a Proctor test can be performed, the choice of which depends upon grain size and the distribution of grains within the soil.

Method A: 4-inch mold: Material retained on a No. 4 sieve is discarded. When the amount of material retained is 7% or greater, Method C should be used instead.

Method B: 6-inch mold: All other requirements like Method A.

Method C: 6-inch mold: Material retained on a 3/4-inch sieve is discarded. When the amount of material retained is 10% or greater, Method D should be used instead.

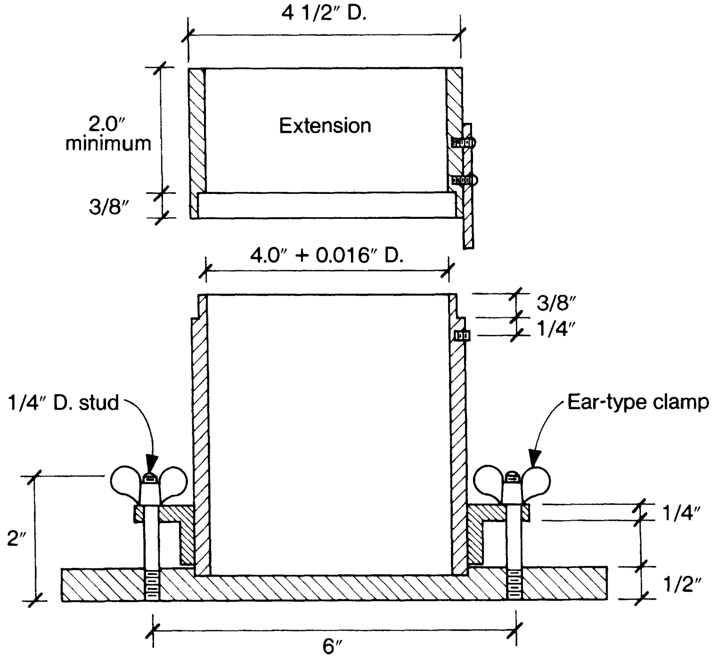


FIGURE 12-9. Proctor Density Test apparatus. [Ref. 2]

Method D: 6-inch mold: Material retained on the 3/4-inch sieve is placed on a 3-inch sieve. Material retained on the 3-inch sieve is discarded. Material passing the 3-inch sieve but retained on the 3/4-inch sieve shall be replaced with an equal amount of material passing a 3/4-inch sieve but retained on a No. 4 sieve. The replacement material shall be taken from the unused portion of the sample.

Before performing the test, a sufficient quantity of representative soil shall be obtained for testing. The moisture content of all material shall then be reduced until the material can be easily crumbled between the fingers. It should be noted that a small amount of water may remain in the sample.

The test shall be performed on a series of at least four samples. These samples shall be prepared by adding increasing amounts of water to each sample so that the moisture content will vary from one sample to another by approximately 1.5%.

Each sample shall be placed in the mold with collar attached and compacted in three layers approximately equal in height. Sufficient material must be included in the third layer to ensure that the compacted soil will project somewhat above the mold into the collar. The rammer used for compaction shall weigh 5.5 pounds and shall free fall a measured distance of 12 inches. Each layer of soil shall receive 25 blows of the rammer when using the 4-inch mold and 56 blows when using the 6-inch mold.

After compaction, remove the collar and trim the sample, even with the top of the mold. The mold and the sample contained within it is now weighed. Remember, this is a moist sample. The mold and sample are then placed in an oven, and all the moisture is removed. The mold containing the dry sample is then weighed. Because the weight of the mold is known, the weight of the moist and dry samples can easily be determined.

Numerical values for density and moisture content are now computed by use of the following formulas:

$$\text{Moist unit weight} = \gamma_{nat} = \frac{W}{V} = \frac{W_w + W_s}{V} \tag{12-2}$$

$$\text{Moisture content} = w\% = \frac{W_w}{W_s} \times 100\% \tag{12-3}$$

$$\text{Dry unit weight} = \gamma_{dry} = \frac{W_s}{V} \tag{12-4}$$

$$\text{or } \gamma_{dry} = \frac{\gamma_{nat}}{w + 1} \ (w \text{ in decimal}) \tag{12-5}$$

The results of this test can be presented visually by plotting the interaction between dry density and moisture content for each of the samples, as shown in Figure 12-10.

Point 1 is called the optimum moisture content because this is the moisture content at which maximum compaction, hence maximum density, can be

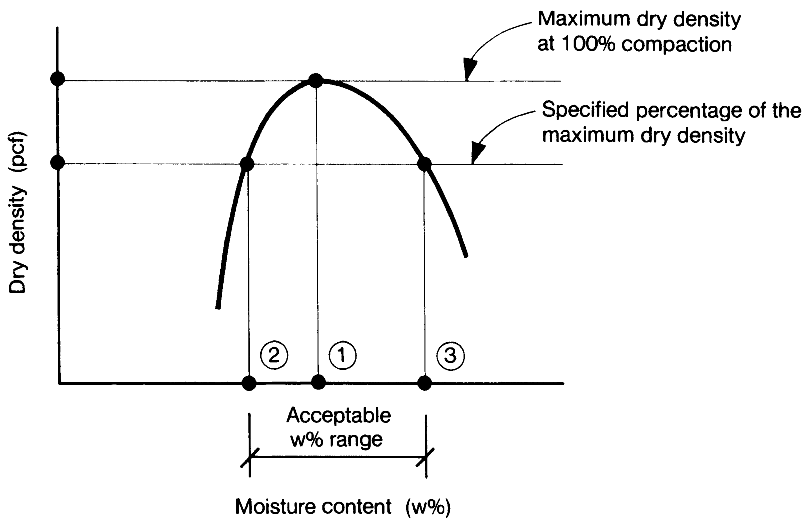


FIGURE 12-10. Curve showing variation in the dry density of a fine grained soil as a function of moisture content.

achieved. Points 2 and 3 identify the lower and upper limits of moisture content within which any specified percentage of the maximum density can be obtained.

12-8. COMPACTION OF MIXED GRAINED SOILS

Natural deposits of soil frequently contain gravel, sand, silt, and clay in various proportions. Such soils are referred to as mixed grained. Soils that are mixed grained will, in all likelihood, exhibit some of the characteristics of both coarse grained and fine grained soils. The presence of measurable cohesion is the deciding factor as to whether a particular soil should be compacted in accordance with coarse grained or fine grained requirements.

1. Soils that do not exhibit any measurable cohesion are treated as coarse grained soil. Compaction is based on relative density D_r.
2. Soils that exhibit measurable cohesion are treated as fine grained soil. Compaction is based on the Proctor Density Test.

12-9. VERIFICATION OF IN-PLACE SOIL DENSITY

Contract Requirements for Soil Density

The architect or engineer in charge of the work must first establish the standard to which the field work must conform. This standard differs, depending upon whether the soil is classified as coarse grained or fine grained. The standard is determined as follows:

1. Standard #1, for coarse grained soil: Specify the minimum relative density. This value can be taken from those given in Table 12-1 or it can be based on the designer's own experience.
2. Standard #2, for fine grained soils: Specify the required minimum dry density. Then determine the acceptable range of moisture content through which this density can be achieved, as verified by the Proctor Density Test, as indicated in Figure 12-10.

Methods of Verification

To verify whether the compacted soil meets the requirements of its particular standard it is first necessary to determine the natural density γ_{nat} of the in-place soil. This soil includes solids, moisture and air.

The natural density can be determined by sending undisturbed samples to a testing laboratory for analysis. This method, however, is time consuming and

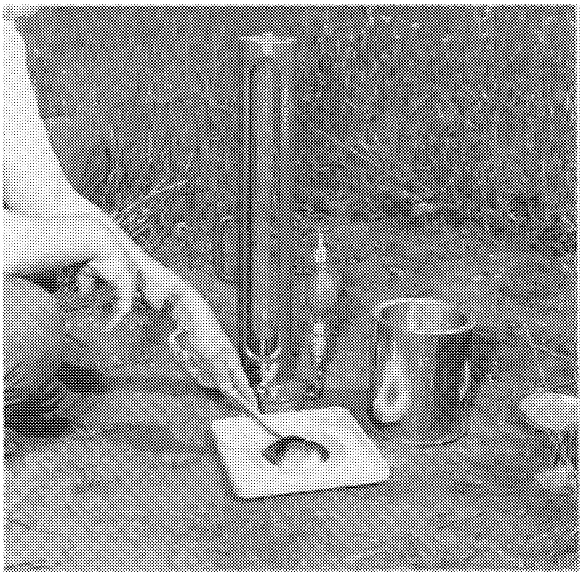


FIGURE 12-11. Soil density by the rubber balloon method. [Ref. 17]

the field work would be delayed while waiting for test results and subsequent approval or correction of the work.

There are three methods whereby the density of the in-place soil can be determined rather quickly in the field. These methods are described in the following ASTM Standards:

1. ASTM Designation D-1556: Density of soil in Place by the Sand-Cone Method.
2. ASTM Designation D-2167: Density and Unit Weight of Soil in Place by the Rubber Balloon Method.
3. ASTM Designation D-2922: Density of Soil and Soil-Aggregate in Place by Nuclear Methods.

These three methods cannot be indiscriminately specified by the architect or engineer because each method has certain requirements which enumerate the characteristics of the soil for which they are best suited.

In the sand-cone method and in the rubber balloon method, shown in Figure 12-11, the idea is to excavate a volume of soil with hand tools. This volume of soil is then weighed, after which the volume of the hole is carefully measured. Knowing the weight and volume of the sample, the in-place density of the soil may easily be computed, using Formula (2-1).

The nuclear method is illustrated in Figure 12-12. This method provides a rapid, nondestructive technique for the determination of in-place moist soil density. Test results may be affected by chemical composition, heterogeneity and surface

FIGURE 12-12. Soil density using nuclear methods. [Ref. 4]

texture of the material being tested. The techniques also exhibit a spatial bias in that the apparatus is more sensitive to certain regions of the material being tested. Nuclear methods, of course, pose special hazards and require special care. The work must be done in strict conformance with all safety requirements and must be performed only by trained personnel.

Regardless of which of the three methods are used to determine the in-place density, it should be remembered that the dry density is what is ultimately required. The dry density can be determined once the moisture content of the soil is known. Procedures for the determination of moisture content are described in the next article.

12-10. FIELD CONTROL OF MOISTURE CONTENT

There are two general methods whereby moisture content can be determined.

1. Accurate results can be achieved by the laboratory analysis of undisturbed samples in accordance with the following ASTM Standard:

   ASTM Designation D-2216: Laboratory Determination of Moisture Content of Soil, Rock and Soil-Aggregate Mixtures

This method, however, takes time, and it has been noted that time is sometimes a luxury that the work in the field can ill afford.

2. Very quick and reasonably accurate results can be obtained in the field with the Speedy Moisture Tester, conforming to AASHTO Designation T-217, and illustrated in Figure 12-13. Because it requires only about 3 minutes to perform this test, the moisture content can be monitored almost continuously.

It is important that the moisture content of the soil be maintained as close to the optimum moisture content as can reasonably be expected during all stages of the compaction process. When the soil is too dry, the moisture content can be increased by sprinkling water over the surface, and thoroughly mixing it into the soil so as to produce a uniform moisture content throughout the mass. When the soil is too wet, the moisture content can be reduced by spreading the soil out and letting it dry in the sun. No soil should be compacted until its moisture content has been accurately determined and approved by the architect or his representative.

The moisture content of the soil should be determined in the field on an ongoing basis with the Speedy Moisture Tester. Additionally, spot checks of the moisture content of freshly compacted soil should be made by the testing laboratory in accordance with the D-2216 procedure.

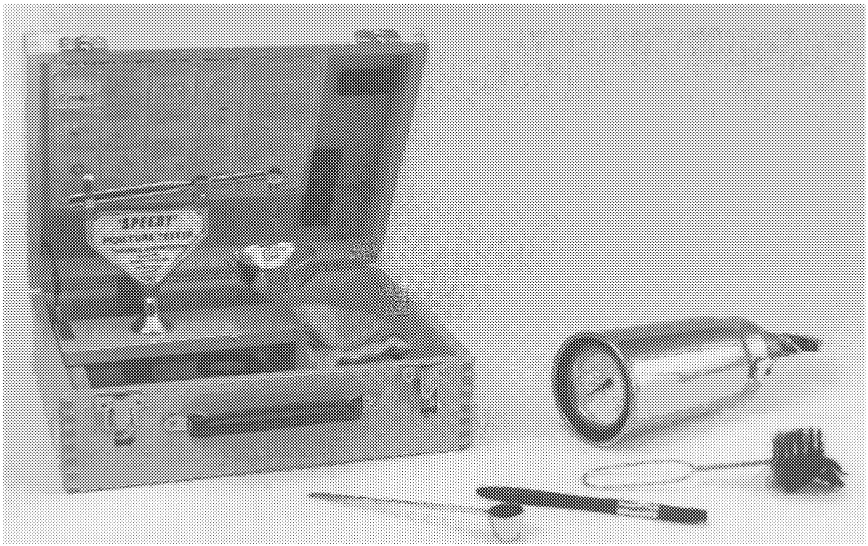


FIGURE 12-13. Speedy Moisture Tester for determination of moisture content of soil in the field. [Ref. 4]

12-11. COMPACTION CHARACTERISTICS OF USCS SOIL GROUPS

Table 12-3 provides valuable information relative to the use of various soil groups as building materials. The six headings in the table correspond to the descriptions given in the following six items:

1. USCS Symbol: Soil groupings in accordance with the Unified Soils Classification System, as indicated in Figure 1-7.
2. Dry Weight: This is the usual range of density that will be realized provided that compaction procedures meet specifications.
3. Compaction: Some soils compact better than others. The results may be good, fair, or poor. The equipment listed has been proved to work best for that particular group of soils.
4. Embankments: When used in the construction of an embankment, the soil has been graded in terms of stability of the slopes.
5. Foundations: The potential bearing capacity for each group of soils has been rated as good, fair, or poor.
6. Settlement and Expansion: This column evaluates the long-term settlement and expansion characteristics of the various soil groups. This evaluation clearly indicates the superiority of coarse grained soils over fine grained soils whenever long-term settlement or expansion due to the availability of free water are matters of concern.

12-12. SAMPLE PROBLEMS

Example 12-1

Required: To compute the relative density of the soil previously analysed in Example 2-3 by using the procedure specified in Article 12-6.

The following weights were determined in Examples 2-1 and 2-3.

	Loosely Packed	In-Place	Tightly Packed
Step 1: V	1 CF	1 CF	1 CF
Step 3: W_s	87.6 #	105.0 #	113.3 # (dry weight)

The relative density is found from Formula (12-1). Note that dry weights are used:

$$D_r = \frac{105.0 - 87.6}{113.3 - 87.6} \times \frac{113.3}{105.0} \times 100\% = 73.0\%$$

Note: This answer agrees numerically with that previously determined in Example 2-3 when void ratios were used in the calculations.

TABLE 12-3. Compaction Characteristics of USCS Soil Groups [Ref. 22]

1	2	3	4	5	6
GW	125-135	Good: tractor, pneumatic or steel-wheeled roller	Very stable	Good	Almost none
GP	115-125	Good: tractor, pneumatic or steel-wheeled roller	Reasonably stable	Good	Almost none
GM	120-135	Good, with close control: pneumatic or sheepsfoot roller	Reasonably stable	Good	Slight
GC	115-130	Fair: pneumatic or sheepsfoot roller	Fairly stable	Good	Slight
SW	110-130	Good: tractor	Very stable	Good	Almost none
SP	100-120	Good: tractor	Reasonably stable	Good to poor	Almost none
SM	110-125	Good, with close control: pneumatic or sheepsfoot roller	Fairly stable	Good to poor	Slight/ medium
SC	105-125	Fair: pneumatic or sheepsfoot roller	Fairly stable	Good to poor	Slight/ medium
ML	95-120	Good to poor, with close control: pneumatic or sheepsfoot roller	Poor stability	Very poor	Slight/ medium
CL	95-120	Fair to good: pneumatic or sheepsfoot roller	Stable	Good to poor	Medium
OL	80-100	Fair to poor: sheepsfoot roller	Not suitable	Not suitable	Medium/ high
MH	70-95	Poor to very poor: sheepsfoot roller	Poor stability	Poor	High
CH	75-105	Fair to poor: sheepsfoot roller	Fair stability	Fair to poor	High
OH	65-100	Poor to very poor: sheepsfoot roller	Not suitable	Not suitable	High
PT		Not suitable for any construction purposes			

1. USCS symbol
2. Dry weight
3. Compaction characteristics
4. Use in embankments
5. Bearing capacity
6. Settlement/expansion

See article 12-11 for further description of headings.

Example 12-2

Required: To determine the in-place unit weight, the dry unit weight and the water content of a particular sample of soil analysed as part of a Proctor Density Test.

Given: Combined weight of mold and moist soil = 9.45 #
Combined weight of mold and dry soil = 8.91 #
Weight of empty mold = 5.38 #
Volume of mold = 1/30 CF

Calculations:

Weight of moist soil = 9.45 − 5.38 = 4.07 #
Weight of dry soil = 8.91 − 5.38 = 3.53 #
Weight of water = 4.07 − 3.53 = 0.54 #

From Formula (12-2): $\gamma_{nat} = \dfrac{4.07}{1/30} = 122.1$ pcf

From Formula (12-3): $w = \dfrac{0.54}{3.53} \times 100\% = 15.3\%$

From Formula (12-4): $\gamma_{dry} = \dfrac{3.53}{1/30} = 105.9$ pcf

or Formula (12-5): $\gamma_{dry} = \dfrac{122.1}{0.153 + 1} = 105.9$ pcf

Example 12-3

Required: To plot the variation in dry density and moisture content as determined during a Proctor Density Test, to determine from this curve the maximum density to which this soil can be compacted, and to determine the optimum moisture content corresponding to that density. Lastly, to determine the range of moisture content through which a relative density of 98% can be achieved:

A Proctor Density Test was performed on six representative samples of soil. The dry density and moisture content of each sample, as determined during this test, were recorded as follows:

Sample number	1	2	3	4	5	6
Dry density (pcf)	112.1	114.9	118.2	118.5	116.1	113.2
Moisture (%)	6.1	7.6	10.0	14.1	16.4	17.9

These values were then plotted on a density–moisture curve, as shown in Figure 12-14. The following information is then read from the curve:

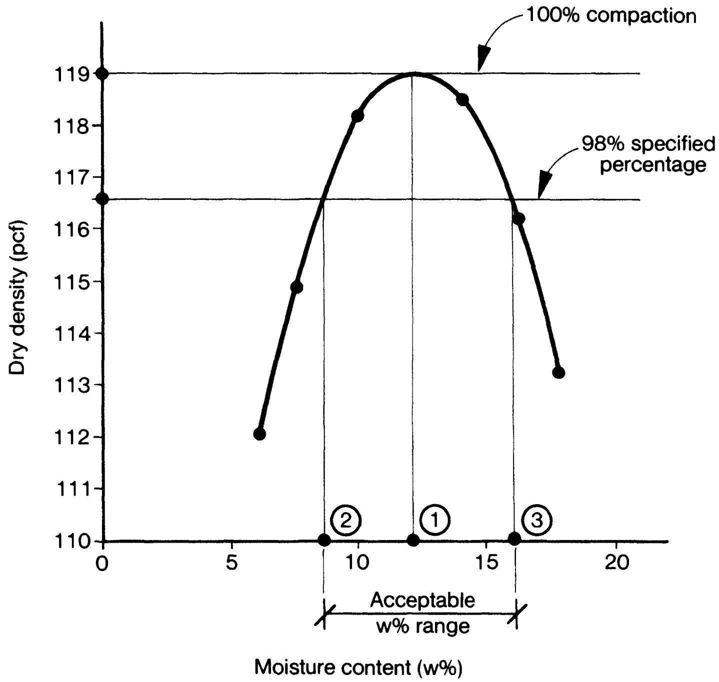


FIGURE 12-14. Example 12-3—Plot of the Proctor Density Test results.

1. The maximum density is 119 pcf at 100% compaction.
2. The corresponding moisture content is 12.1%.
3. A relative density of 98% results in a unit density of 0.98 × 119 = 116.6 psf, as plotted on Figure 12-14.
4. The acceptable range of moisture content for 98% compaction, as read off the curve, is 8.7 to 16.1%.

Example 12-4

Required: To compute the in-place density of a granular soil on which a sand-cone test was performed. Then to compute the dry density, using the results of a Speedy Moisture Test.

Given: (a) The combined weight of sand contained in the funnel of the sand-cone device and in the test hole = 2.06 #
(b) Weight of the sand contained in the funnel = 0.75 #
(c) Unit weight of sand used in test = 95.1 pcf (from lab.)
(d) Weight of in-place soil recovered from test hole = 1.72 #
(e) Moisture content of in-place soil as determined by a Speedy Moisture Text = 14.6%

Calculations: (f) Weight of sand required to fill test hole:

$$2.06 - 0.75 = 1.31 \ \#$$

(g) Volume of test hole $= \dfrac{1.31}{95.1} = 0.01377$ cf

(h) Unit weight γ_{nat} of soil $= \dfrac{1.72}{0.01377} = 124.9$ pcf

(i) Using the relationship that $W = W_s + W_w$, and substituting $W_w = W_s$ times the % of moisture, Then

$$1.72 = W_s + 0.146\ W_s \text{ from which } W_s = 1.50 \ \#$$

(j) And the dry density $\gamma_{dry} = \dfrac{1.50}{0.01377} = 108.9$ pcf

13

Expansive Clay

13-1. INTRODUCTION

General

Expansive clay is a generic term used by architects, engineers and contractors to indicate any soil that exhibits, by observation or by tests, the characteristic of volumetric expansion or contraction when subjected to an increase or decrease in moisture content. This phenomenon occurs only in clays, but does not occur equally in all clays.

Expansive clay is sometimes referred to as an expansive soil, a swelling soil, or a shrink-swell soil. These terms, however, have a somewhat different meaning in that they imply a mixed-grained soil which contains such a high percentage of expansive clay that the clay imparts its own special characteristics to the mass.

Any discussion relative to the change in volume associated with expansive clay must first be preceded by a discussion of clays in general, and un understanding of clay is dependent on an understanding of a particle called a colloid.

Colloids

Certain very small particles carry a surface charge of static electricity. When the particle is extremely small, less than 0.002 millimeters (mm), its surface area is very large compared to its mass. The influence of the electric charge on the comparatively large surface of the particle will then be significantly greater than the influence exerted by gravity on the relatively small mass. This condition is referred to as the colloidal state, and any such particle is called a colloid. The characteristics of a colloid are profoundly affected by the electrical charge carried

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

on the surface of the particle. Extensive research has determined that colloidal particles consist primarily of clay minerals.

13-2. CLAY MINERALS

Structure

Clay minerals are primarily the end product of the chemical weathering of feldspathic rock. Chemically, these minerals are essentially hydrous aluminum silicates, although occasionally the aluminum atoms are replaced with atoms of other elements, such as magnesium, iron, potassium, or sodium. The atomic structure of a clay mineral is highly complex and consists of a variety of combinations and arrangements of two basic building blocks called the silica tetrahedron and the alumina octahedron. These building blocks are diagramatically illustrated in Figure 13-1.

The various building blocks that make up a clay mineral are arranged in orderly sheets, much like the pages of a book. The particular arrangement and chemical composition of these blocks determines the type of clay mineral and its general characteristics.

Interaction With Water

The most important characteristic of clay, from the viewpoint of architectural engineering, is the interaction between the clay particles and water. There are four kinds of water with which the particle interacts:

1. Structural water: This term refers to water in the form of hydroxyl ions (OH) and is an integral, indivisible part of the chemistry of the clay particle.
2. Interlayer water: This term refers to a very thin sheet of water which occurs in some, but not all, of the different kinds of clay minerals. For those clays

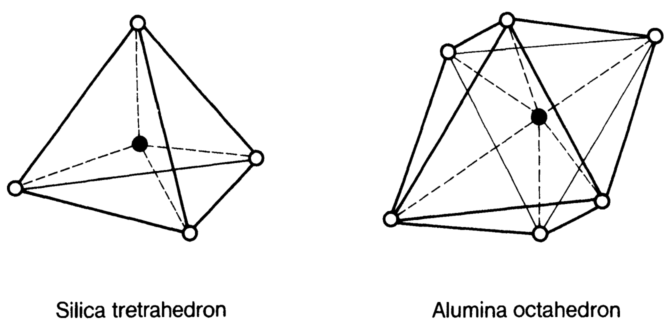


FIGURE 13-1. The basic building blocks of clay minerals, in which the open circles (○) indicate oxygen atoms or hydroxyl ions, and the filled circles (●) indicate a silica or aluminum atom, or a replacement atom.

in which it does occur, it forms a separation between the sheets of the basic building blocks. This water is also an integral, indivisible part of the chemistry of the clay particle.

3. Free water: This is water that is contained within the voids of the soil. It is frequently referred to as pore water.
4. Absorbed water: This is water that is bonded to the surfaces and edges of the particle by the electric charge characteristic to all colloidal particles. This water is also sometimes referred to as pore water.

Clay particles interact with water because of the electrical charge which exists on their surface. The intensity with which the particle attracts and holds the water molecules as absorbed water depends upon the particular mineralogy and morphology of the particle. Clay minerals are sometimes described as having a high or a low surface activity. It is important to understand that the chemistry of the clay determines to a considerable extent the ease or difficulty with which free water moves within a clay deposit or within any mixed-grain soil deposit that is rich in clay. Clay minerals having a high surface activity will more readily capture and absorb any available free water and will more slowly release this absorbed water during periods of dry weather or drought.

13-3. MAJOR CLAY GROUPS

Clay minerals are grouped according to chemistry, interaction with water and use. The three main groups of clay are Kaolinite, Illite, and Montmorillonite, as briefly described in the following paragraphs.

Kaolinite: $Al_4\ Si_4\ O_{10}\ (OH)_8$

The kaolinite group of clays, of which the mineral kaolinite is the principle member, are the most prevalent of all clays. A kaolinite mineral is composed of two sheets, one consisting of silica tetrahedrons and the other of alumina octahedrons. These sheets are very strongly bonded together. Kaolinite, therefore, is very stable and has little tendency to change volume when exposed to water or to drought. Kaolinite contains no interlayer water because of the way the sheets fit together. It does, however, have the ability to absorb sufficient water to develop plasticity.

Kaolinite is used extensively in the ceramics industry and in the manufacture of fire brick because of its excellent firing properties.

Illite

The illite group of clays does not have a principle mineral. Instead, this name refers to a group of mica-like clay minerals. The basic structural unit of an illite clay is composed of two silica tetrahedral sheets with a central octahedral sheet. Potassium is the primary element in the central sheet. Illite exhibits more plasticity than kaolinite, but has little tendency to change volume when exposed to a change in moisture content unless there is a deficiency in potassium.

Illites are used extensively in the manufacture of brick and tile.

Montmorillonite: $Si_8\ Al_4\ O_{20}\ (OH)_4 \times nH_2O$

Montmorillonite is a group name for clay minerals that have expansive structures, and is also the principle mineral of the group. The structure of montmorillonite consists of an alumina sheet held between two silica sheets to form a weakly bonded, three sheet layer. This mineral exhibits considerable variation in characteristics because of the interchange between elements within each sheet. Iron or aluminum, for example, may replace the aluminum in the alumina sheet, and aluminum may replace some of the silicons in the silica sheet.

When considered in the context of the design of the foundations for a building or other structure, this mineral exhibits the highly undesirable characteristic of undergoing considerable change in volume when moisture is added to or deleted from the soil mass. This characteristic can lead to very serious problems of heaving or of settlement. Buildings and other structures should never be founded on soils rich in montmorillonite. Even the suspicion of the existance of this type of clay at a construction site should be a signal for the need of an in-depth analysis of the soil characteristics.

In other industries montmorillonite has some redeeming qualities. It is widely used in the petroleum industry as a clarifier and decolorizer of lubricating oil, and is also used as a bonding agent in the production of moulding sand.

Bentonite

Bentonite is a member of the montmorillonite family of clays, and is composed of minerals which are the result of the alteration of volcanic ash. Bentonite deserves special recognition because it is the most notorious of the expansive clays. Bentonite responds quickly and with considerable change in volume to any inflow of water. The properties of bentonite vary widely from site to site and frequently vary within the same site. This makes its presence very destabilizing wherever it is found. Bentonite is very unpredictable, and the amount of volumetric change cannot be realistically determined by laboratory analysis.

The very fact that bentonite responds so quickly to the addition of water makes it very useful in the following instances:

1. As a waterproofing agent: Bentonite is marketed commercially as a prepackaged material that is placed beneath slabs on ground to control the transmission of moisture or of actual water through the slab. The underlying principle is that the Bentonite will swell in response to the inflow of ground water and will provide a watertight seal between the subgrade and the slab.
2. As a slurry to prevent cave-in: In the construction of drilled piers it has been the general practice to case the hole with a steel shell whenever drilling

through soft, unstable material which might collapse into the hole. Modern methods have shown that when a thick slurry of bentonite is pumped into the hole while the auger is being advanced it will prevent the side walls from caving in. The reason for this is that the bentonite will expand hard against the side walls, providing lateral resistance against their collapse. Later, when the pier is concreted, the concrete must be deposited at the bottom of the shaft and worked upward in a continuous pour so as to displace the bentonite slurry. During this operation the concrete is pumped through a flexible tube called an elephant trunk. The lower end of the trunk must be held below the surface of the concrete at all times during the pour to make certain that no bentonite will be trapped within the pier.

13-4. CATION EXCHANGE

Additional understanding of the interaction between clay particles and water will be obtained through an introduction to the concept of ions and cation exchange.

Ions

Ions are atoms, or groups of atoms, which have been electrically altered through the gain or loss of one or more electrons. When an atom is electrically neutral the number of positive charges (protons) and negative charges (electrons) is equal. Any loss of electrons will leave the atom with a positive charge, and the resulting ion is called a cation.

As a general rule, cations are the end product of the alteration of metallic atoms. Negative ions, called anions, are generally the result of the alteration of nonmetallic atoms.

The most prevalent cations are sodium (Na), potassium (K), calcium (Ca), magnesium (Mg), and iron (Fe). These cations exist in solution in the pore water of the soil. Cations in close proximity to the clay particle are attracted to it and electrically held. The bond between the particle and the cations, however, is not necessarily permanent and these cations can be replaced by other cations. This condition is known as cation exchange.

All clay minerals exhibit the ability of cation exchange. This is why all clays, to some degree, are expansive clays.

The ability of a clay particle to exchange cations is dependent on the type of clay mineral. The table which follows indicates the approximate ratio of cation exchange ability between the three most prevalent minerals:

Kaolinite	1 (taken as base)
Illite	3 times the ability
Montmorillonite	10 times the ability

It has been previously noted that all clay particles carry a surface charge of static electricity. This charge is negative on the surfaces and positive around the edges. The surfaces of the clay particle, because of their negative charge, attract cations from the pore water. These surfaces also attract the positive end of free water molecules. The negative end of other water molecules are attracted to the cations which are being held on the surface of the particle and they are also attracted to the positively charged edges of the particle. This process of opposites attracting opposites is an effort, on the part of all the players, to achieve electrical equilibrium. The result is a veritable mishmash of particles, ions and water molecules.

The attraction for cations and water molecules, as exhibited by the clay particle, extends out beyond the particle, but with ever-decreasing intensity. Water molecules close to the particle are held with strong electrical bonds. This is the water that is referred to as absorbed water. At some distance away from the particle the electrical bonds have diminished to such an extent that the water is no longer bonded to the particle. This is the water that is known as free water or pore water.

13-5. PARTICLE SIZE

Atoms, ions, molecules and clay particles are tiny things, and can only be studied through the use of an electron microscope. This level of smallness is measured in angstroms, symbolized by the letter Å.

1 millimeter (mm) = 0.001 meter m
1 micron (μm) = 0.001 mm
1 nanometer (mu) = 0.001 μ = 0.000,001 mm
1 angstrom (Å) = 0.1 mu = 0.0001 μ = 0.000,000,1 mm

By way of comparison, the human eye can normally see a particle having a thickness of approximately 0.060 millimeters, or 600,000 angstroms. A silica tetrahedron and an alumina octahedron both measure approximately 5.0 angstroms in thickness. A particle of kaolinite, containing two sheets, has an approximate thickness of 7.5 angstroms. A water molecule, containing one atom of oxygen and two atoms of hydrogen, would fit into a rectangular box measuring approximately 1.0 by 1.0 by 2.0 angstroms.

13-6. ATTERBERG LIMITS

The characteristics whereby a soil may increase or decrease in volume belong exclusively to clay. A mixed-grained soil, however, will also exhibit these characteristics, provided that the soil consists predominantly of clay. The potential for volumetric change of any soil can be determined quantitatively by performing a series of relatively simple laboratory tests on a remolded sample of the soil. Remolded soil is used for this test because the consistency (Article 2-10) of the sample must be progressively altered, and it has been previously determined that the consistency of a remolded sample can be changed at will by increasing or decreasing the water content. Remolding, it will be remembered, is a term used to indicate the physical manipulation of a previously undisturbed sample by kneading and working it in the hands.

To prepare the sample for this test the remolded clay is mixed with sufficient water to produce a thick, liquid suspension. The water content of this mixture is then gradually reduced, and the physical changes in the samples are continuously and very carefully monitored. As the water content is reduced the mixture will gradually pass from a liquid state into a plastic state and then finally into a solid state. The water content at each change in state is recorded. Remember, water content, which is analogous to moisture content, was previously defined as:

$$w\% = (W_w / W_s) \times 100\%$$

The various stages through which the sample will pass during this test and the water contents which must be recorded are briefly described in the paragraphs which follow. This concept of correlating physical properties of soils with water content was introduced in 1911 by a German engineer named Atterberg, and it is in his honor that these properties are called Atterberg limits.

Liquid Limit (*LL*)

The water content at which the mixture passes from liquid to plastic is called the liquid limit, identified as the water content at which the sample begins to exhibit a small, but measurable strength in shear. The test shall be performed in accordance with the following ASTM Standard:

> ASTM Designation D-4318: for Liquid Limit, Plastic Limit and Plasticity Index for Soils

Plastic Limit (*PL*)

A sample in the plastic state can be rolled into long, thin threads and, when suspended from the fingers, will support its own weight. The water content at which the mixture passes from plastic to solid is called the plastic limit, and is identified as the water content at which the sample begins to crumble and can no longer be rolled. The plastic limit test is performed in accordance with the same ASTM Standard as is the liquid limit test. A test in progress, in which a technician is rolling a sample into a thread, is illustrated in Figure 13-2.

Shrinkage Limit (*SL*)

When the water content in the sample is gradually reduced below that of the plastic limit the sample will continue to shrink, but the shrinkage will be less

FIGURE 13-2. One of the Atterberg tests in progress.

and less. The water content at which further loss of water does not result in a reduction of volume is called the shrinkage limit. This limit must be performed in accordance with the following ASTM Standard:

ASTM Designation D-427: for Shrinkage Factors of Soils

These three tests are relatively easy to perform on relatively simple equipment. They must, however, be performed by experienced personnel, working in the controlled conditions of a testing laboratory. Performed carelessly, these tests could give erroneous results, leading to entirely erroneous conclusions.

The relationship between the Atterberg Limits and the approximate consistency of the remolded clay sample can be illustrated diagrammatically, as shown in Table 13-1.

Plasticity Index (*PI*)

The numerical difference between the liquid limit and the plastic limit is called the plasticity index, symbolized by *PI* or I_p. This number is extremely important in the field of soil mechanics because it gives insight into the expansion or contraction characteristics of any particular soil. These characteristics can be evaluated from the information given in Table 13-2.

It should be noted that soils that exhibit change in volume with change in water content are usually referred to as swelling soils. Remember, however, that

TABLE 13-1. Atterberg Limits for Remolded Clay

Consistency	Physical State	Atterberg Limits	
Very soft	Liquid		
Soft		Liquid limit LL	Plasticity Index PI
Stiff	Plastic		
Very stiff		Plastic limit PL	
Hard	Semi–solid		
		Shrinkage limit SL	
	Solid		

such soils can also exhibit shrinkage, since shrinkage is merely the reversal of swelling.

Clays having a low swelling potential, PI <10, exhibit relatively small change in volume when subjected to a change in moisture content. These are the more stable of the clays and are less likely to damage structures having sufficient flexibility to withstand the small amount of anticipated differential settlement.

Clay layers having a high swelling potential, PI >20, can exhibit considerable change in volume when subjected to a change in moisture content. These are the clays that can cause extensive damage to any structure by reason of excessive vertical, horizontal or differential movement. Any attempt to build on these clays or to attempt to confine them against free expansion should not be seriously considered.

The thoughtful evaluation of any soil having a medium plasticity index is of critical importance. This evaluation is made by comparing the actual water content of the in situ soil with the boundaries of the plastic range as determined by the Atterberg tests. A water content near the liquid limit would indicate a soil that has considerable potential to shrink, but none to swell. A water content near or below the plastic limit would indicate a soil having considerable potential to swell, but none to shrink. A soil with a water content at or near the median of the plastic range may shrink or swell, depending upon future change in the water content of the surrounding soil which forms the environment of the soil in question.

TABLE 13-2. Swelling Potential of Clay Soils as Indicated by Plasticity Index

Plasticity Index PI	Swelling Potential	Percent Increase
0	Nonplastic	—
1–5	Slight	—
5–10	Low	<1.5
10–20	Medium	1.5–5.0
20–40	High	5.0–25
>40	Very high	>25

The plasticity index of a given clay soil, and hence the ability of that soil to shrink or to swell, is primarily a function of the following three factors:

1. Which mineral group is the more prevalent within the soil
2. Which cation is the more prevalent within the mineral
3. The strength with which the structure of the particles are bonded

Kaolinites are the least active of all the clays, exhibiting *PI* values in the usual range of about 10 to 20. Illites represent the mid-range of potential activity, with *PI* values of approximately 30 to 60. The most active of all the expansive clays are the montmorillonites. These clays have a demonstrated wide range of high *PI* values, even the low end of which exceeds that of the illites. With sodium as the prevalent cation, *PI* values in excess of 600 have been recorded.

Clays exhibiting a low plasticity index are referred to as lean clays, while those with a high plasticity index are referred to as fat clays.

13-7. OTHER TEST PROCEDURES

It is important to know the detailed characteristics of any soil intended for use in a building. This is particularly true for those soils known to have or suspected of having swelling characteristics. The information given in Table 13-2 can be used as a guideline, but more definitive information can be determined by the performance of additional laboratory tests.

These tests are based on one of two responses of a swelling soil to an increase in moisture content:

1. If the soil is volumetrically unrestrained so that it is free to expand then it will do so.
2. If the soil is volumetrically restrained so that it is not free to expand then it will exert a bursting pressure on whatever is restraining it.

The tests referred to are the unrestrained swelling test and the swelling-pressure test, both of which are briefly described in the following paragraphs.

Unrestrained Swelling Test

This test is for the purpose of quantitatively determining the amount of swelling that will occur when a particular soil has unlimited access to free water and is permitted to expand without restraint. The apparatus, as illustrated in Figure 13-3, is constructed so as to permit unrestrained swelling vertically while restraining the sample against swelling laterally.

After the initial height of the sample has been carefully measured, an unlimited amount of free water is made available to the sample. The soil reacts by absorbing water and expanding. A point is reached, however, when the sample becomes saturated, and will no longer accept water. It is at this point that the sample ceases to expand. The test is then completed by expressing the vertical expansion

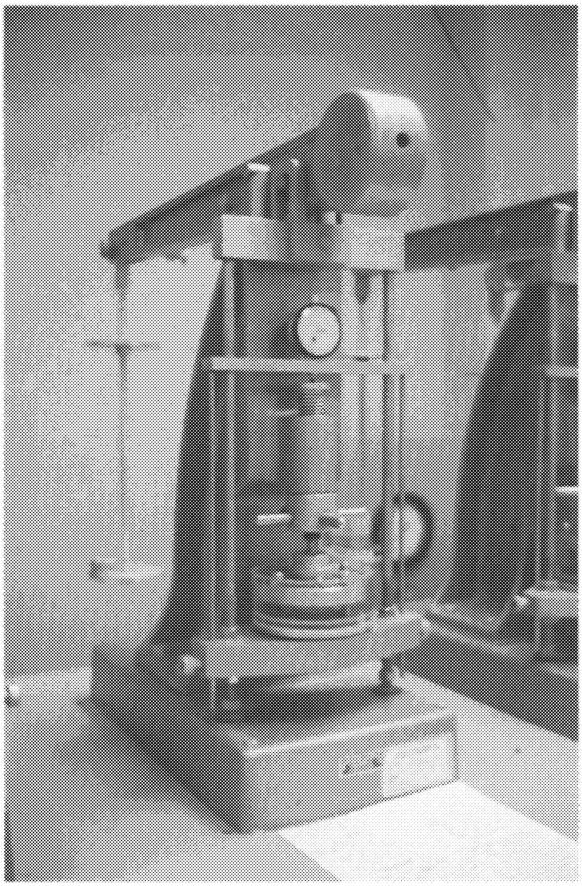


FIGURE 13-3. Swelling test apparatus.

of the soil as a percentage of the original height. The results of this test can be used to approximate the amount of swelling which may occur in a given layer of this particular soil.

Swelling-Pressure Test

This test is for the purpose of determining the amount of pressure that will be developed by a particular soil having unlimited access to free water but confined so that expansion cannot occur.

This test is performed in an apparatus similar to that used for the unconfined swelling test. The equipment is designed, however, to prevent vertical as well as lateral expansion, and is also designed to record the magnitude of any buildup of internal pressure. When free water is introduced to the soil it will react by absorbing the water and will attempt to expand. Since expansion cannot occur, the soil will build up an internal pressure. This pressure, quantitatively, is analogous to the force per unit of area that a swelling soil will exert in the field when it is prevented from expanding. These pressures can be formidable, and on occasion have been known to exceed 20 tons per square foot.

13-8. FACTORS AFFECTING WATER CONTENT

It should be clearly understood that even though a particular soil may have a high potential to shrink or to swell it will remain volumetrically stable unless and until there is a change in the moisture content of that soil. Conditions which would cause a change in moisture content can be divided into the following categories:

Climate and Weather

1. The evaporation of moisture from within the clay layer due to a severe period of low humidity or drought
2. An influx of water due to extended periods of rainfall combined with inadequate drainage or runoff

On-site Construction and Landscaping

3. Extensive excavation within the site, which would expose large areas of underground to the loss of water through evaporation
4. Construction of an on site building, after which the rain water is collected and discharged into a public storm sewer system—this will have the long-term effect of promoting a loss of moisture in the soil because this water will never be replenished
5. Construction of an on site building, after which the rain water is collected and distributed onto the site—This will have the short term effect of reducing the moisture content beneath the building while increasing it beyond the building. The length of time that it will take for moisture stabilization to occur is a direct function of the permeability of the soil.
6. The migration of water from the site to adjacent low ground, or to the site from adjacent high ground
7. The close proximity of large trees or foliage, which draws more water from the ground than that which is returned by rainfall or by the physical addition of water by sprinkling

Off-site Construction

8. Nearby excavation, of either a temporary or permanent nature, which would allow for the migration of water away from the site

9. Nearby construction, which will have the effect of removing a certain amount of ground surface from absorbing its share of rain fall or releasing its share of moisture through evaporation
10. Installation, at an adjacent site, of a tile drainage system for dispersement into the ground of industrial or residential effluent

Unforseen Problems

11. Leaks in underground piping

13-9. BUILDING CONSTRUCTION

General

This article refers to public, private, and commercial building construction, as opposed to one- or two-unit residential house construction. All structural elements of the building and all other elements which are an integral part of the architectural design of the building or of the site, must be protected against the adverse action of expansive soil. It is the opinion of the author that these elements should not be constructed on expansive soil, except as noted in the following exception.

An exception to the above rule could be considered if a method were devised whereby the moisture content of a particular area of expansive soil was permanently prevented from change. It would then be permissible for certain elements, such as interior slab on ground and large areas of exterior paving, to be constructed on expansive soil provided (a) that the plasticity index of the soil does not exceed ten, and (b) that the soil exhibits suitable load bearing characteristics. The fact that a soil is an expansive soil is not in itself a problem. The problem is that it is practically impossible to insure that the moisture content of a given soil mass will remain permanently constant.

Construction of Foundations

The main building foundations must always be isolated from the expansive soil, even if it is thought that the moisture content can be maintained without change.

When the soil is confined to a relatively thin layer, and when there is good bearing soil at a reasonable depth beneath it, the quickest and most cost effective approach is to excavate through the expansive soil and to install spread footings. When the layer of expansive soil is relatively thick, or when good bearing cannot be found within a reasonable depth beneath it, then deep foundations, such as piles, piers or caissons must be used. The proper choice of foundation will depend on the depth required to reach good bearing and on the material through which the foundation must be extended. Alternate methods are discussed in detail in other sections.

Construction of Slab on Ground

There are five ways by which slab on ground is commonly constructed in the type of building described in this article. There are basement slabs, ordinary slabs on ground, slabs over crawl space, self supporting one-way slabs with beams and self-supporting two-way slabs.

1. Basement Slabs: Basement slabs are usually constructed for the purpose of housing mechanical services, storage and similar things, and to provide unlimited access to all mechanical work serving the upper areas. Basement floors are generally placed well under ground, usually about ten to twelve feet below the exterior grade. In most cases, therefore, the basement slab will be well below the zone of expansive soil. The slab, then, can be designed as an ordinary slab on ground, in accordance with the recommendations in Appendix D.

2. Ordinary Slabs on Ground: It is generally anticipated that the first floor slab of buildings without basements will be cast directly on the ground, examples of which are shopping centers, building supply houses, carpet and furniture stores, and automobile sales and service areas. It is this slab which must be protected from rise or fall due to expansion or contraction of underlying soil.

The proper way with which to eliminate this problem is to eliminate the offending soil. When this soil occurs in only a relatively thin layer, and is accessible to earth moving machinery, then that layer of soil may be cost effectively removed and replaced with acceptable borrow fill. This slab, then, can be designed as an ordinary slab on ground, in accordance with the recommendations of Appendix D.

When the layer of offending soil is considered to be too thick, or when the soil beneath does not have sufficient bearing capacity, then one of the following methods should be considered.

3. Slabs Over Crawl Space: This method provides for the construction of a self supporting structural slab over a crawl space or plenum beneath the slab. This crawl space is relatively shallow, being on the order of approximately three feet in the clear. This floor is self-supporting. Therefore, it must be designed and constructed as a typical reinforced concrete system of slabs, beams, piers and foundations.

This type of construction not only assures structural integrity but offers the additional advantage of simplifying installation and future access to all the required mechanical services. This method requires the use of wood shores and formwork, just like any other suspended floor system would require. In this regard, it is important that the architectural specifications require the removal, after construction, of all mudsills, shores and formwork.

Specifications should also require that the surface of the crawl space be protected with a concrete slab. The purpose of this slab is twofold:

1. To provide a work area that will be essentially clean of debris
2. To protect the crawl space from the influx of vermin

This slab, which is sometimes called a mud slab, is not a structural slab in the true sense of the word. For this reason it is relatively thin, two to four inches in thickness, and lightly reinforced, sometimes with welded wire mesh, sometimes with none at all.

4. Self-Supporting One-Way Slabs With Beams: In this method, the beams (usually called grade beams) and slabs are poured directly on hollow cardboard forms manufactured especially for this particular purpose. The use of these forms is illustrated in Figure 13-4. These forms are manufactured to disintegrate quickly when exposed to moisture. The principle behind their use is that they will attract available moisture from the soil, and then rot out. This has the ultimate effect of leaving the floor slab completely independent of volumetric changes within the subgrade.

This method of construction has several inherent problems, including:

1. Slabs must be designed as self supporting units to span between the grade beams. Grade beams must be designed to span between the pile or pier foundations.
2. Maintaining the integrity of the surface of the subgrade prior to and during installation of the cardboard forms.
3. Installation of reinforcing, and maintaining it in proper position prior to and during the pour.
4. Maintaining the side walls of the grade beams prior to and during the pour.

These objections could at least be partially overcome by constructing the floor in two separate pours. The first pour would include all of the grade beams up

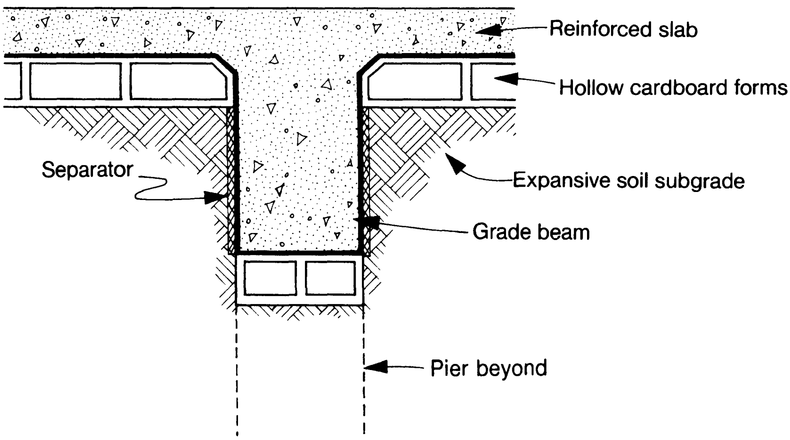


FIGURE 13-4. Construction of slab on ground with hollow cardboard forms.

to the underside of the slab. The second pour would be that of the slab. When this system of two separate pours is used it may be necessary to measure the effective depth of the grade beam from the underside of the slab, thereby resulting in a deeper grade beam. This would be a design decision, based on the method of interlock between the two pours.

Alternate: When there is sound bearing within a reasonable depth below subgrade the grade beams of item 1 can be deepened to bear on this sound bearing, thereby acting as continuous bearing walls. Pile or pier foundations are, therefore, no longer required. When using this alternate, the cardboard forms beneath the grade beams must be omitted.

5. Self Supporting Two-Way Slabs: As an alternate to the use of one-way slabs and grade beams the slab could be designed as a two-way flat plate cast on hollow cardboard forms and spanning between piles or piers placed approximately on 8- to 12-foot centers. This method would eliminate all grade beam excavation and formwork other than around the perimeter of the building. The slab would normally be 1 to 2 inches thicker than the one-way slab design. Particular care must be taken in the design of perimeter shear (punching shear) around the supporting pile or pier foundations.

13-10. RESIDENTIAL CONSTRUCTION

General

Residential construction, unfortunately, for reasons of time or money or ignorance, is rarely given the thoughtful consideration that should be required for construction in and around soils consisting of expansive clays.

Some houses are built with basements or crawl spaces. This automatically alleviates any problem associated with differential movement caused by a change in moisture content. The reasons for this are as follows:

1. Basements usually extend about 8 to 10 feet below grade. This will normally be well below the zone of moisture change.
2. Crawl spaces in residential construction are usually about three feet in clear height between the first floor joists and the mud slab. The first floor is constructed of conventional wood framing and is posted down to good bearing, bypassing any problems with expansive clay.

Effect of Moisture Change

Before the construction of a residence, the moisture content throughout the immediate area, in all probability, is in equilibrium. Due to the extreme slowness with which moisture migrates through soils rich in clay, this equilibrium of moisture will continue to exist for some time after construction has been com-

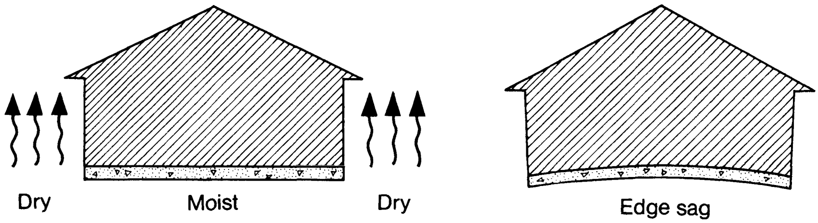


FIGURE 13-5. Residential edge sag, due to an extended period of dry weather.

pleted. During this time, the ground beneath the slab will be stable, and there will be no tendency on the part of the slab to lift or sag. There will come a time, however, when the ground beyond the residence, whose surface is exposed to atmospheric conditions, will either lose moisture through evaporation or gain moisture through rainfall. Since the ground beneath the slab cannot lose or gain moisture in the same way, there will be a loss of equilibrium between the two areas. The soil adjacent to the perimeter of the residence will shrink or swell according to its loss or gain in moisture. The effect that this has on the slab on ground is illustrated in the details which follow.

Condition 1: This condition is representative of the effect of an extended period of dry weather. The soil beyond the residence is exposed to the atmosphere and will lose moisture through evaporation. The soil beneath the slab is insulated against the loss of moisture through direct evaporation. Although there will be an effort, on the part of the soil, to regain equilibrium through the migration of moisture, the immediate effect is that the soil around the perimeter of the slab will shrink and the edge of the slab will sag. This condition is known as edge sag, and is illustrated in Figure 13-5.

Condition 2: This condition is representative of the effect of an extended period of wet weather. The soil beyond the residence is exposed to the atmosphere and will gain moisture from rainfall. The soil beneath the slab is insulated against the gain of moisture from direct rainfall. Although there will be an effort, on the part of the soil, to regain equilibrium through the migration of moisture, the immediate effect is that the soil around the perimeter of the slab will swell and the edge of the slab will lift. This condition is known as edge lift, and is illustrated in Figure 13-6.

The preceding illustrations of edge sag and edge lift give the impression that this is a rather straightforward problem. Nothing could be further from the truth. Soils rarely respond to an outside stimulus in a straightforward way. This is particularly true when the effect of this stimulus is to produce a migration of moisture in a soil that is rich in clay. Evaporation, rainfall and run-off are rarely the same around the entire perimeter of a residence. Sections of perimeter may

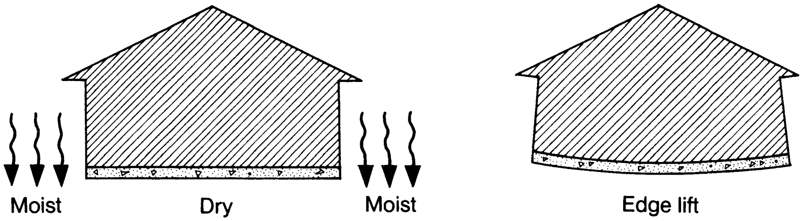


FIGURE 13-6. Residential edge lift, due to an extended period of wet weather.

shrink or swell differently than other sections. Differential lift or sag around the perimeter of a residence can be disastrous, as illustrated in Figure 13-7.

This process of gain or loss in moisture in the soil, with a corresponding lift or sag in the slab, can be endlessly repeated. A slab with edge sag, for example, can gain sufficient moisture to evolve into a edge lift condition. The same slab, in the months to come, can lose moisture and revert to an edge sag condition. It is not at all unusual for these slabs to react to seasonal changes, or to more immediate changes, for example, how frequently one waters or does not water the lawn.

Effect of Moisture Migration

The migration of moisture can have a dramatic and deleterious effect on a residence whose slab on ground foundation is built on expansive soil. Migration can occur in either of two directions, as noted:

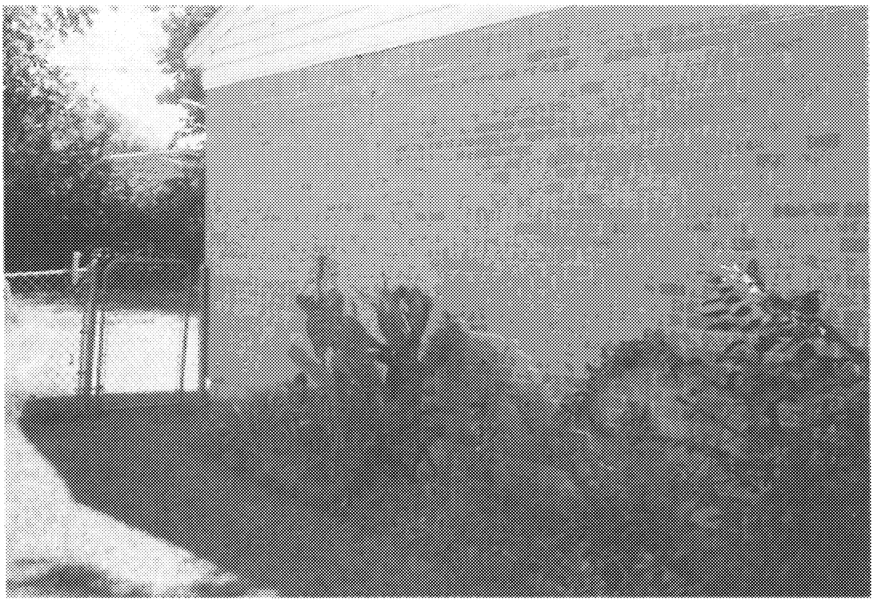


FIGURE 13-7. Damage to a residence, as caused by differential settlement.

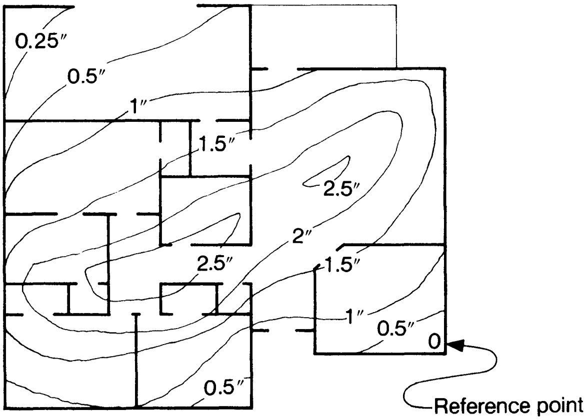


FIGURE 13-8. Variation in slab topography as caused by the migration of moisture beneath the slab.

1. From the soil beneath the slab to the soil beyond the slab
2. From the soil beyond the slab to the soil beneath the slab

Remember:

1. The migration of moisture will only occur when the moisture content of the soil in two adjacent areas is not in equilibrium.
2. Moisture always migrates from a moist area to a less moist area, never the reverse.
3. The migration of moisture from one area to another will be very slow because of the very low permeability of a soil rich in clay. The movement will be somewhat faster, however, as the difference in moisture content is more pronounced.

Damage due to the migration of moisture is not necessarily limited to exterior walls, but may occur anywhere throughout the slab. The inherent flexibility of the slab, along with a change in moisture content in the supporting soil, combine to create a condition of differential movement over the entire slab. This movement can cause extensive damage to the residence. The author has seen residences whose variation in slab topography exceeded 4 inches. An example of the type of variation that can be found in topography of a typical residence is shown in Figure 13-8.

13-11. RECOMMENDATIONS FOR RESIDENTIAL CONSTRUCTION

The proper way to construct the foundations and slab on ground of a house situated in expansive soil is as itemized below. Anything else is a compromise.

1. Extend all foundations to sound bearing beneath the expansive soil, using piers, if necessary.
2. Do not cast the slab directly on expansive soil. If the layer of expansive soil is relatively thin it should be removed and replaced with acceptable, compacted borrow fill. If the layer of soil is too thick to reasonably remove, then isolate the slab from the soil using one of the following methods:
   (a) Provide a crawl space arrangement as described in Article 13-10.
   (b) Install self-supporting one-way or two-way reinforced concrete slabs, as described in Articles 13-9(4) and 13-9(5).

Although anything else is a compromise, certain compromises are listed herein not as a means of eliminating the effects of a shrinking or swelling soil but in an attempt to limit them so as to avoid excessive damage.

1. Foundations around the perimeter of the residence should consist of a concrete turndown poured monolithically with the slab on ground. This foundation shall extend a minimum of 24 inches below finished grade, as shown in Figure 13-9. Reinforcement from the slab shall extend into the turndown. Provide additional reinforcing as indicated in the figure. Designed in this way, it is the intention that these foundations shall act as a perimeter buffer against moisture migration into or out of the soil beneath the slab.
2. Determine the moisture content of the subgrade and of the adjacent ground just prior to the scheduled pouring the slab. Site grading and excavation may have upset the balance in moisture content between these two areas. If the moisture content of the subgrade is lower than that of the adjacent ground then the subgrade should be prewetted. If the moisture content is higher than the adjacent ground then construction of the slab on ground

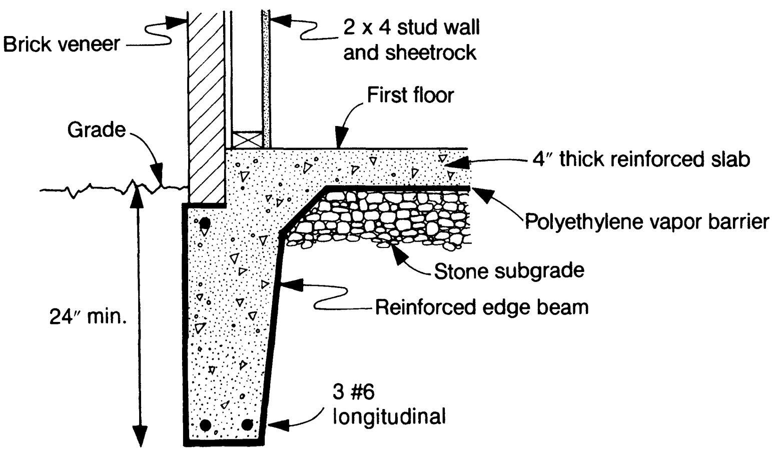


FIGURE 13-9. Typical edge beam detail for a residential slab on ground.

should be delayed so that the subgrade can dry out. The idea here is to equalize, as best one can, the moisture contents of the subgrade and adjacent ground so there will be little or no tendency for water migration from one area to another.

3. Stabilization of subgrade may also be accomplished by the addition of hydrated lime, $C_a(OH)_2$. This alternate can prove to be very effective if the soil and lime are properly mixed, installed and allowed to stabilize. The chemistry behind the inclusion of lime is that it will lower the plasticity index of the soil, thereby reducing the affinity of the clay particles for water. The reader is cautioned that hydrated lime can be very dangerous because of severe burns which can result when used improperly. For additional information on the use of lime as soil stabilization see Krebs and Walker, [Ref. 10], page 231.
4. Provide a gentle but positive slope to the finished grade so that excess surface water will be directed away from the residence.
5. Large trees and heavy foliage draw a considerable amount of water from the soil. Tree roots frequently extend beneath the slab on ground in their search for water. Lawns having large trees or heavy foliage require considerable water. When there is insufficient rainfall, or when the evaporation is too great, an auxiliary system of sprinklers should be installed.
6. For areas without large trees or heavy foliage, but where auxiliary water is required, it may be satisfactory to install leak hose around the perimeter of the residence. This area can then be watered if the ground becomes dry or hard or if it shrinks away from the foundation.
7. Finally, post-tensioning of the slab on ground is becoming more and more popular in regions of expansive clays. In order to be effective, however, this work must be designed by engineers and installed by skilled technicians. General guidelines for the design of post-tensioned slabs on ground are given in Appendix D. The thought behind post-tensioned construction is that the slab will act as a unified, solid structural element having increased resistance to differential settlement. The slab used in post-tensioning work will be thicker than those usually used in residential construction. A minimum thickness of 5 to 6 inches is recommended. Although post tensioning can produce good results, the author has observed a number of builders who had little idea of the complexities of this type of construction, so that the work was simply not done properly. The advantages that post-tensioning could produce, therefore, would not be realized.

It has been the author's experience that foundations built on expansive soil sage more readily than lift. Once a foundation has sagged because of the shrinkage of the supporting soil, the addition of water does not necessarily result in lifting the foundation back to its original level. It would seem, therefore, that the adage of "an ounce of prevention is worth a pound of cure" is very applicable in residential construction. The fact that home repair is one of the big businesses in areas of expansive soil is mute testimony to the fact that the majority of builders do not use that ounce of prevention.

13-12. INSPECTION FOR EVIDENCE OF RESIDENTIAL DAMAGE

The following guidelines may be used to determine whether a residence has sustained damage due to the shrinking or swelling of an expansive soil. When the damage is severe it will be self-evident. There will be very noticeable cracks and separations in walls, partitions and ceilings. Doors and windows will not open and close easily, or they may not operate at all. There may also be a noticeable pitch to the floor.

Lesser, more subtle damage may be identified by making the following observations:

1. Check for straight lines in horizontal lines that should be straight, for example: brick joints, eave and ridge lines, siding, exposed edges of foundations. If any of these lines curve or bend this indicates that the foundations have subsided or heaved.
2. If the horizontal lines appear to be straight then use a transit to determine whether they tilt from one end to the other.
3. The tilt of horizontal lines can also be checked by filling a very thin, flexible hose with water and applying the principle that water seeks its own level. This test is also very effective in checking for variations in the elevations of floor slabs in different parts of the house, particularly from room to room when separated by partitions.
4. Use a carpenter level to check whether walls are out of plumb, particularly corners.
5. Look for cracks in brickwork or in plaster, particularly those that vary in thickness.
6. Check the joint where the chimney meets the brickwork. If the joint becomes wider as it goes up then there is an outward tilt to the chimney.
7. Check windows and doors to see if they work properly. If they rub or do not close completely this indicates that they are out of square.
8. All trim should fit tightly against the wall or ceiling. Look for gaps, particularly at interior and exterior corners. Any movement in trim work may be evidence of movement within the foundations.
9. Note whether the earth has pulled away from the foundation wall. This is not necessarily evidence that damage has occurred. It is, however, positive proof of shrinkage of the soil and is certainly indicative of potential trouble. This condition can be alleviated by installing a leak hose around the perimeter of the building and allowing it to trickle water during periods of high moisture evaporation

10. Examine the lawn. Evidence of separation (giving the appearance of an alligator's back) is clear indication that the soil has dried out and is in need of water.

13-13. RELEASE OF OVERBURDEN

Soils rich in clay occasionally produce strange happenings during construction. Such a happening occurred on one of the author's projects.

One day the contractor excavated a rather large area to a depth of about ten feet. It was his intention to place reinforcement and pour concrete the next day. When he arrived at the site the next morning he found a lake. After several frenzied telephone calls, which resulted in a trip to the site by the architect and engineer, the contractor was told to wait. After a wait of several days the water disappeared, the excavation dried out, and the work could continue. What happened?

The soil being excavated was rich in clay, and this particular clay consisted primarily of minerals having a relatively low surface activity. Such a clay has little ability to attract and hold free water. This soil had been compressed for a long period of time by the weight of the soil above. When this overburden was removed the compressed soil began to rebound, and the free water seeped into the excavation from the sides and percolated up into the excavation from beneath, hence a lake. After several days of hot, dry weather most of the exposed water evaporated into the air and the remainder seeped back into the ground. The contractor had a free lesson in one of the many different responses that a clay rich soil can make whenever long term, existing conditions are changed in any way.

14

Characteristics of Rock

14-1. GENERAL

The geologist is interested in information such as the origin, history and other more highly technical characteristics of a given rock mass. The architect and engineer, however, are usually interested only in those characteristics which will enable them to predict the performance of the in situ rock as a suitable material upon which to bear the foundations of their building or other structure. Their interest, then, is to identify the specific kind of rock, to determine the existence and extent of jointing and weathering within the rock mass, and to determine the presence, source and elevation of ground water.

Sound rock is an excellent material upon which to bear a building foundation because it is a very stable material and exhibits practically no compression under load. This means that a building founded on solid rock will not settle. Not all rocks have the same degree of hardness, however, and allowable bearing pressures must reflect the capacity of the rock to withstand the weight of the foundations without crushing.

14-2. GENERAL CLASSIFICATIONS OF ROCK

Classification by Origin

All rock originated as the result of volcanic action, either on the surface of the earth or within the Earth's crust. In the years that followed many rock masses were subjected to a variety of natural forces which altered their original characteristics. There are three main classifications of rock:

C. I. Duncan Jr., *Soils and Foundations for Architects and Engineers*

Igneous: This term applies to rock that has remained basically unaltered after its initial solidification.

Metamorphic: Rock in this classification is the result of physical alteration due to geological heating and/or geological pressure, usually within the Earth's crust. The result is a solid mass of denser, harder, more crystalline material.

Sedimentary: Rock composed of cemented fragments of other rocks which have been transported from their source and deposited elsewhere by wind or water and formed into sheetlike layers by physical or chemical action.

Some common names of rock as classified by origin are as follows:

Igneous	Metamorphic	Sedimentary
Basalt	Gneiss	Limestone
Diabase	Marble	Sandstone
Granite	Schist	Shale
	Serpentine	
	Slate	

Classification By Texture

Rock is also classified by texture, which relates to the size, shape and arrangement of the constituent elements in sedimentary rock, and to the crystallinity, granularity and fabric of the constituent elements of igneous and metamorphic rocks. There are four different textural classifications, as follows:

Interlocking: Interlocking rock consists of crystals that have been interwoven into a fairly homogeneous mass during solidification. Rock of this classification exhibits similar properties in all directions.

Cemented: This type of rock consists of individual grains of single or multiple minerals which have been joined together by means of chemical action. In general, cemented rock exhibits fairly similar properties in all directions.

Laminated: This characteristic is the result of rock fragments having been deposited into thin layers during sedimentation. Laminated rock has strongly directional properties.

Foliated: Foliated rock, as with laminated rock, has strongly directional properties. This characteristic is due, however, to alteration by heat and pressure rather than to sedimentation.

Examples of rock classified by texture are as follows:

Interlocking	Cemented	Laminated	Foliated
Basalt	Sandstone	Limestone	Gneiss
Diabase		Shale	Schist
Granite			Serpentine
Marble			Slate

14-3. FAULTS IN ROCK MASSES

Any mass of underlying rock may contain defects because no rock mass is completely solid. Joints, fissures, and other defects occur in all types of rock formations and for a variety of reasons. Sedimentary rock, for example, is actually deposited and formed in sheetlike layers with joints between each layer. These joints, which are called bedding planes, represent a weakness in the rock mass along which fracturing may occur.

Rock masses, particularly those near the surface of the Earth's crust, are occasionally subjected to physical disturbance such as rock slide or earthquake. This kind of activity can cause fracturing within the rock mass and in the case of more violent agitation it may cause large volumes of rock to break apart. Such rock is called fractured or broken rock. Badly fractured or broken rock is usually considered to be unsuitable as a bearing material. Such rock is usually removed and the excavation extended down to suitable bearing. It should be noted that in most cases where badly fractured rock is found at the surface there have been tectonic movements in or around that locale. In areas of dense fracturing, it can be anticipated that further shifting of the rock may occur, with possibly catastrophic damage to any structure built upon it. Some understanding, therefore, of the geological features of the area should prove helpful to the architect in choosing the optimum site for the construction of the building.

Joints represent a potential plane of weakness within the rock mass since little or no cohesion exists across the joint. Of significance, then, is the direction and thickness of the joints since they may determine the stability of a site. Horizontal joints are usually of more consequence than vertical joints, because they interfere with the transmission of the building loads into the rock below. Inclined joints can represent a serious defect because of the loss of shearing strength along the joint. Joints may be open or tight. Open joints are usually filled with a variety of mineral deposits, including decomposed rock and clay. The thickness of the joint is of particular concern, primarily because of the increased compressibility and instability of the weaker material within the joint. In areas of high water table joints act as a waterway through which water can migrate or flow. All these factors should be taken into consideration by the designer during the preliminary stage of the project.

14-4. WEATHERING

The surface of any rock, when exposed to the air or to water, may experience physical alteration or chemical decomposition. This condition is known as weathering, and such rock is called weathered, decomposed, or rotted rock. The incidence of weathering usually is in evidence near the surface of the rock formation, but can also occur wherever there are fissures or other defects within the rock mass; thus, weathering can occur at substantial depths below the surface of the rock.

Physical alteration may be due to expansion or contraction, or to the abrasive action of wind or water. Freezing water can be particularly damaging because of the increase in volume produced when water changes into ice. Rocks subject to physical weathering retain the same mineralogical composition as the original rock. Chemical decomposition may be due to oxidation, hydration or carbonation. The latter condition is particularly injurious, and occurs when carbon dioxide in the air combines with rain water to produce carbonic acid. Chemical decomposition of rock results in a change in the composition of the original rock and in the formation of new minerals.

14-5. CORE BORINGS

General

The characteristics and safe bearing pressure of underlying rock at the site of a proposed building should be determined by engineers experienced in the evaluation of such matters. In order to make this evaluation it will be necessary to take core borings at the site and to perform a laboratory analysis on the recovered samples. This work is performed under contract with the owner as a part of the overall subsurface soil exploration for the project.

Core borings in rock are made with a machine driven rotary drill. The drill bit is of very special construction because its purpose is not only to cut into the rock but is also to recover rock samples at any desired depth below the surface. The rock sample is called a core. Core diameters are usually 1-3⁄16″ or 2-1⁄8″, depending on the size core specified. The larger core, although more expensive, is considered to be more reliable and is recommended for explorations involving major building construction.

Immediately upon recovery, cores are match marked and placed in a core box for transport to a testing laboratory for analysis and evaluation. Typical core boxes are illustrated in Figures 3-13 and 14-1.

Laboratory Evaluation

Cores are first examined visually to determine the general characteristics of the rock. The existence and extent of fissures, joints, weathering and other defects affecting the load carrying capacity of the rock can be determined by this examination.

A laboratory test will be conducted to determine the unconfined compression strength of the rock. This test is performed on a carefully trimmed rock sample whose height is 1-½ to 2 times the diameter of the core. This test is very similar to the compression test performed on concrete cylinders. In the performance of this test it is very important that the sample is properly trimmed so as to have both ends cut at right angles to the length of the sample. The numerical value

FIGURE 14-1. A typical core box used to transport rock cores. [Ref. 1]

of the unconfined compression strength on rock is usually stated in tons per square foot and is symbolized by the letter (q_u).

The unconfined compression strength is found to vary considerably between different kinds of rock and between different deposits of the same kind of rock. Sound rock, however, is a very strong material, and deposits of sound rock offer excellent support for any almost any kind of superimposed lateral or vertical load. In buildings or other structures that are sensitive to settlement, foundations are frequently extended down to rock because the compression of the rock, and therefore its settlement, is virtually nonexistent.

Rock is frequently found to be much stronger than concrete, and values in excess of 2400 tsf have been recorded. As a comparison between the relative strengths of rock and concrete the strength of 4000 psi concrete is computed in terms of tsf:

$$\frac{4000\ \#}{1\ \text{si}} \times \frac{1\ \text{ton}}{2000\ \#} \times \frac{144\ \text{si}}{1\ \text{sf}} = 288\ \frac{\text{tons}}{\text{sf}} = 288\ \text{tsf}$$

For additional information relative to core borings, refer to Article 3-8.

14-6. ROCK QUALITY DESIGNATION

Valuable insight as to the consistency and load carrying capacity of the rock mass can be obtained by determining a property called the rock quality designation

(RQD) from the core samples. The RQD is commonly called the recovery ratio and numerically equals the collective length of core recovered divided by the length of core drilled.

The standard length of core barrel is five feet. This limits the core recovery at any particular depth to five feet. Cores, however, are rarely recovered in 5-foot lengths. In computing the RQD, only sound pieces of rock are used; those less than 10 cm long (4″), are excluded, as are those which are broken or fragmented.

RQD is 100% for strong and massive rocks which yield cores whose pieces are all longer than 10 cm, and is near 0% for jointed rocks [Ref. 12]. Rock quality designations are given in Table 14-1.

TABLE 14-1. Rock Quality Designation [Ref. 12]

RQD %	Quality Designation
90–100	Excellent
75–90	Good
50–75	Fair
25–50	Poor
<25	Very Poor

14-7. ALLOWABLE BEARING PRESSURE

General Considerations

Factors that must be considered in the determination of the allowable bearing pressure on rock, symbolized by q_a, are as follows:

1. An evaluation of all the general factors known about the site, including the extent and classification of the rock
2. The existence and extent of jointing and weathering, all of which increase the compressibility of the mass, thereby reducing the safe load carrying capacity (This information is obtained from a careful examination of the rock cores and any loose fragments or decomposed material brought to the surface during the test boring procedures. The incidence of jointing and weathering, and its effect on the safe bearing pressure, is very closely related to the RQD values.)
3. Determination, by laboratory analysis, of the unconfined compression strength of the rock q_u (This property, which is analogous to the ultimate compression strength of concrete, is determined in accordance with the following ASTM Standard:)

ASTM Designation D-2938: Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens

The unconfined compression strength of rock as determined in the laboratory is always greater than that corresponding to the same rock as it exists in nature. For this reason this property of rock is of importance only in the case of very soft, weathered, or decomposed rock.

4. Requirements as set forth by the governing building code

Bearing Pressure From RQD Values

The compressibility of rock is closely related to the characteristics of the joints which occur in the rock. When the joints are tight, or when they are no wider than a fraction of a inch, the compressibility of the rock mass is reflected by the Rock Quality Designation. The allowable bearing pressure, q_a, as based on the RQD, is given in Table 14-2.

Note: In no case shall the allowable bearing pressure as given in the table exceed the unconfined compression strength, q_u, as determined by laboratory analysis.

TABLE 14-2. Allowable Stress on Rock [Ref. 16]

RQD %	q_a (tsf)
100	300
90	200
75	120
50	65
25	30
0	10

Bearing Pressure From Code

All building codes contain a section that specifies the allowable bearing pressure to be used for different classifications of rock and soil. In many codes these requirements are very conservative. In order to avoid undue penalty on major construction higher values are usually permitted when they can be substantiated by test and engineering analysis.

In no case shall the allowable bearing pressure, as determined by any other source or analysis, exceed that which is specified by the applicable building code without written approval of the building official in charge of granting exceptions to the code.

14-8. NEW YORK CITY BUILDING CODE

Some building codes are so well written, and their engineering provisions so well researched, that they can be used as an excellent source of engineering information. One such code is the New York City Building Code [Ref. 6], condensed highlights of which are given below:

1. Hard Sound Rock—Includes crystalline rocks such as Fordham gneiss, Ravenswood gneiss, Palisades diabase, Manhattan schist. Characteristics are: the rock rings when struck with a pick or bar; does not disintegrate after exposure to air or water; breaks with sharp fresh fracture; cracks are unweathered and less than ⅛ inch wide, generally no closer than 3 feet apart; core recovery with a double tube, diamond core barrel is generally 85% or greater for each 5 foot run—60 tsf.
2. Medium Hard Rock—Includes crystalline rocks of (1) above, plus Inwood marble and serpentine. Characteristics are: all those listed in (1) above, except that cracks may be ¼ inch wide and slightly weathered, generally no closer than 2 feet apart; core recovery with a double tube, diamond core barrel is generally 50% or greater for each 5 foot run—40 tsf.
3. Intermediate Rock—Includes rocks of (1) and (2) above, plus cemented shales and sandstone of the Newark formation. Characteristics are: the rock gives a dull sound when struck with a pick or bar; does not disintegrate after exposure to air or water; broken pieces may show weathered surfaces; may contain fractured and weathered zones up to 1 inch wide spaced as close as 1 foot; core recovery with a double tube, diamond core barrel is generally 35% or greater for each 5-foot run—20 tsf.
4. Soft Rock—Includes rocks of (1) (2) and (3) above in partially weathered condition, plus uncemented shales and sandstones. Characteristics are: rock may soften on exposure to air or water; may contain thoroughly weathered zones up to 3 inch wide but filled with stiff soil; core recovery with a double tube, diamond core barrel is less than 35% but not less than 20% for each 5 foot run, but standard penetration resistance in soil samples is more than 50 blows per foot—8 tsf.

14-9. ROCK GROUTING

There are occasions when it is desirable to fill the fissures and voids occurring within a particular volume of rock. This can be done with varying degrees of success by the process of grouting. The kind of grout to be used will have a cement or chemical base, depending upon the specific circumstances. Exposed fissures and voids can usually be filled by allowing the grout to flow freely into the voids. When it is desired to fill voids that are beneath the surface, it will first be necessary to drill holes in the rock (similar to those of core borings) and then to force the grout into the voids under pressure.

The reasons why the use of grout may be considered are as follows:

1. To increase bearing capacity by filling fissures that must transmit load into the underlying stratas.

FIGURE 14-2. Hampton Road bridge over Interstate 30 bearing on bedrock. [Ref. 21]

FIGURE 14-3. Closeup of arch anchorage into bedrock. [Ref. 21]

2. To provide for additional stability within the rock mass.
3. To provide for the partial control of water which permeates throughout any rock mass below the water table. The control of water should not imply waterproofing since the flow of water cannot be totally prevented.

14-10. BEDROCK

Architects, engineers, and contractors frequently refer to a particular mass of rock as bedrock. What they mean by the use of this term is as follows:

> An essentially solid mass of rock, free of loose pieces or fragments, rock whose surface cannot be penetrated by a pick or shovel, and the surface of which will ring when struck by a pick or a bar.

Bedrock provides not only high resistance to vertical loads, but to lateral loads as well. When the bedrock is in close proximity to the surface of the ground this characteristic of lateral strength allows the engineer the option of using an arch in the construction of a highway or railroad bridge. Arches are a favorite structural system used by engineers to carry heavy loads over long spans. Although the arch may carry only vertical load it is the nature of arch behavior to develop an outward thrust at the base. The ability of an arch to function properly, therefore, is primarily due to the capability of the foundation material to resist this outward thrust. Bedrock is an excellent material for this purpose.

The advantages of arch construction, combined with the vertical and lateral load carrying capability of bedrock, have been used in numerous highway and railroad bridges. An example of this kind of construction is the Hampton Road bridge over Interstate 30 just west of Dallas, as pictured in Figures 14-2 and 14-3.

APPENDIX A

A Discussion of Shear-Friction

A-1. GENERAL CONSIDERATIONS

A typical cold joint to which the concept of shear-friction is applicable is illustrated in Figure A-1. Note that in this detail the surface of the concrete is deliberately roughened between pours. This is an inherent requirement to this concept.

The purpose of this appendix is to examine the feasibility of using the concept of shear-friction to design the two cold joints originally introduced in Article 11-7, and described as follows:

1. The joint between a footing and a wall, as shown in Figure A-2.
2. The joint between a wall and a floor slab, as shown in Figure A-3.

The American Concrete Institute, in Section 11.7 of [Ref. 5], specifies a procedure called shear-friction, which is applicable to the transfer of shear forces

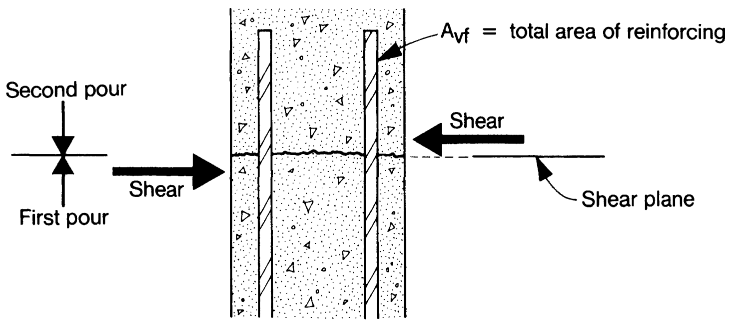


FIGURE A-1. Shear friction at typical cold joint.

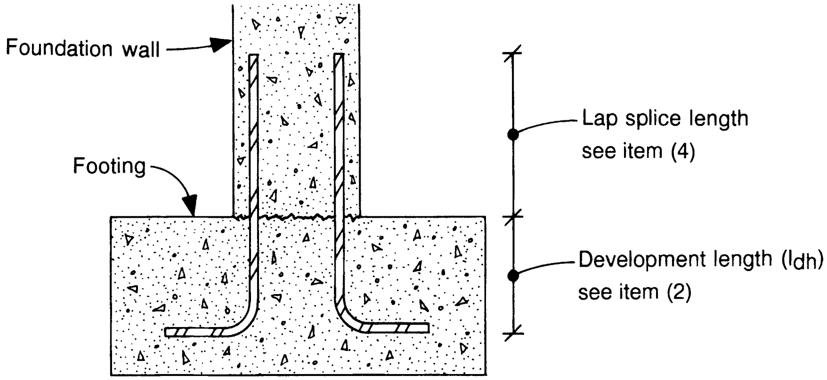


FIGURE A-2. Shear friction at cold joint between footing and wall.

across any plane subject to pure shear, as opposed to diagonal tension. Pure shear exists at the following conditions:

1. At the face of the support of corbels and brackets cast monolithically with their support
2. At the cold joint between two elements that have been cast at different times, as between footings and walls, and between walls and floor slabs

Although the code recognizes the inherent strength of concrete in pure shear, the procedure as applied to corbels and brackets is to assume that the concrete at the face of the support has failed in shear. Based on this assumption, the interlocking effect of the concrete is now ineffective, and the mode of transfer across this plane becomes analogous to that which occurs across a cold joint. The theory which follows is applicable to both of the above noted conditions.

Slippage along the joint must be prevented because a total shear failure would

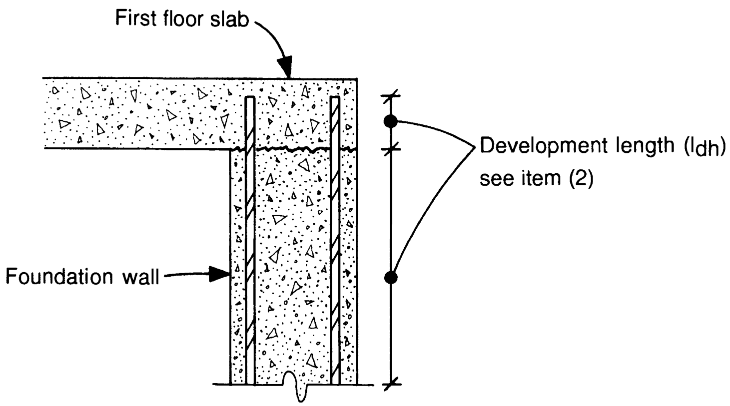


FIGURE A-3. Shear friction at cold joint between wall and floor slab.

inevitably result. Slippage is prevented by the addition of reinforcing steel across the joint. This reinforcing provides a clamping force acting normal to the joint and provides for the development of frictional resistance along the surfaces of contact. In order for the concept of shear friction to be theoretically workable, it must be assumed that a minute slippage actually does occur between the contacting surfaces and that the roughness and irregularity of these surfaces is sufficient to cause a slight separation normal to the joint. This separation, with the accompanying stretch in the reinforcing steel, induces a tensile force of sufficient intensity to develop the required clamping force.

A-2. DEVELOPMENT OF REINFORCING

The shear-friction theory necessitates the tensile development of the required reinforcing on both sides of the cold joint. The lengths required for the tensile development of various reinforcing bars is given in items (1) and (2) of Table E-1.

The main difficulty with achieving a workable design based on the method of shear friction is that of providing the required development length on both sides of the joint.

In Figure A-2, the development length within the footing, item (2), may require that the footing be thickened to accommodate the vertical length of bar. The lap splice length within the wall, item (4), will not be a problem because of ample wall height.

In Figure A-3, the lap splice length within the wall, item (4), will again be no problem because of ample wall height. The development length, item (1), in the floor slab will almost assuredly require more thickness of slab than is available, even with the potential reductions in length, as noted in Article E-2. There are two ways of solving this problem. One is to provide some method of mechanical anchorage, such as used in corbels. The other is to use a shear key. Under normal circumstances the use of a shear key is the better solution.

A-3. CLOSING COMMENTS

When applying the concept of shear-friction to the design of corbels and brackets, the basic assumption is that a crack will develop across the shear plane. If a crack does not develop, this method is invalid and serves only as an additional safety factor to the primary transfer element—which is pure shear acting across the shear plane.

It should be noted that if a crack occurs in waterproof construction it may damage the waterproof seal depending on the type of waterproofing used. A waterproofing membrane would have sufficient flexibility to be undamaged. An ironite rich cement plaster, however, while being an excellent waterproofing agent, is bonded so tightly to the concrete that any crack in the concrete would be expected to transmit through the plaster.

In the case of shear transfer across the cold joint of two elements cast at different times, like the two cold joints under consideration herein, the concept of shear-friction assumes that:

1. There will be a minute slippage across the joint; and
2. There will be sufficient roughness and lifting capability on the adjoining surfaces so that they will be forced apart, and this will cause
3. The reinforcing steel to be stretched, thereby inducing tensile stress, which acts as a clamping force, so that
4. Frictional resistance will be developed along the shear plane.

In the opinion of the author, the concept of shear-friction depends upon too many assumptions, of which several may even be questionable. It is recommended, therefore, that the transfer of shear across these joints be accomplished solely by the use of shear keys, as described in Appendix B.

APPENDIX B

Shear Key Analysis

B-1. TYPICAL SHEAR KEY DETAILS

An examination of the cold joint between the foundation wall and the first floor slab, as illustrated in Figure A-3, clearly shows the impracticability of using the method of shear-friction to achieve the transfer of horizontal earth forces from the wall to the slab. There is simply not enough height on the floor slab side to achieve adequate bar length development. Therefore, another method of transfer must be used. The shear key method is considered to be the most logical choice for the following reasons:

1. Shear keys provide a positive means of load transfer.
2. The calculations by which the safe transfer load is determined are relatively straight forward.
3. Contractors are used to installing shear keys and do so as a matter of course. Certain keys require some special care in formwork, but again, the contractor is knowledgeable in this area.

Typical shear keys are illustrated in Figure B-1. Note that in this detail the side walls of the shear key between the footing and the wall are sloped. Refer to Article 11-7 for a discussion of when these sides can be sloped and when they should be vertical.

B-2. TYPICAL LOAD REQUIREMENTS

In order to have some quantitative idea of the earth pressures that must be transferred, consider the following:

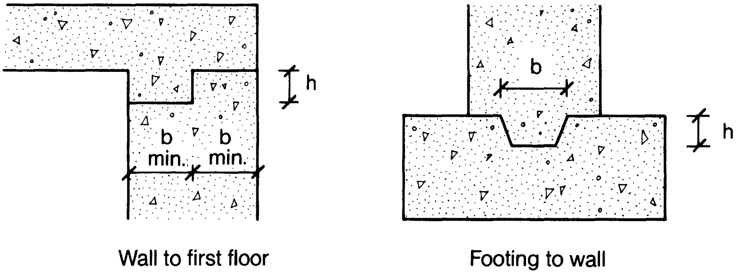


FIGURE B-1. Typical concrete shear keys.

1. The upper and lower transfer forces of a 12 foot high basement wall supporting earth having a density of 125 pcf and a coefficient of active pressure of 0.40 will be 1200 and 2400 pounds per linear foot, respectively, provided that the wall is without surcharge or water pressure. For the related pressure diagram refer to Figure 9-6.
2. If the earth in the preceding example becomes completely saturated, the transfer forces increase to 2100 and 4200 pounds per linear foot, respectively. For the related pressure diagram refer to Figure 9-8.

The shear key used in these transfers bears a resemblance to a corbel or bracket, the main differences being:

1. The length of bearing of a shear key is usually much smaller than that of a corbel or bracket. This has the effect of substantially reducing the bending moment acting on the shear plane.
2. The smaller length of bearing precludes the development of diagonal tension within the shear key. This element, then, is subjected only to pure shear across the shear plane. Corbels and brackets, on the other hand, are commonly designed on the basis of diagonal tension.
3. The transfer loads required of a shear key are usually much less than those carried by corbels or brackets, whose main function is to carry beams or girders.

B-3. SHEAR KEY THEORY OF DESIGN

Note: The design of shear keys is based on one linear foot of key.

Induced Stresses

The American Concrete Institute does not specify a procedure to be used in the design of a concrete shear key. Therefore, one must be improvised.

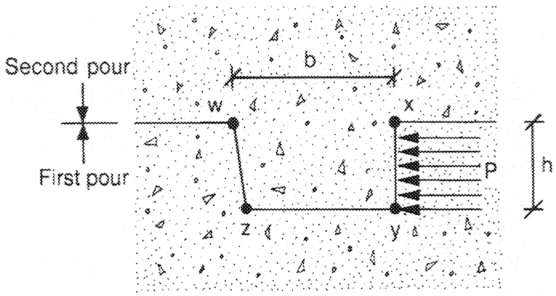


FIGURE B-2. Stress distribution in a shear key.

The shear key must resist stresses induced by bearing, flexure and shear, as illustrated in Figure B-2.

The stresses induced into the shear key by the external load P, and identification of the plane that resists them are as follows:

1. Bearing on the vertical surface of contact, indicated by the line *xy*. The area that resists this bearing is:

$$A = 12\,h$$

2. Flexure on the plane of bending, indicated by the line *wx*. The section modulus which resists this flexure is:

$$I/c = 12\,b^2/6$$

3. Shear on the plane of shear, indicated by the line *wx*. The area that resists this shear is:

$$A = 12\,b$$

Allowable Stresses

Bulletin D85, published in 1965 by the Portland Cement Association, recommends that the design loads transferred through corbels be increased by a factor of 4⁄3. Subsequent publications based on research by other authorities make no reference to this correction factor. For this reason, the factor will not be used in the analysis that follows.

Paragraph B.3.1(c) of the ACI Building Code 318-83 [Ref. 5], specifies the following permissible bearing stress on the surface of contact:

$$f_{brg} = 0.3\,f'_c \text{ (psi)} \tag{B-1}$$

Paragraph 9.5.2.3 of the same code provides insight as to the ultimate cracking moment that can be developed by an unreinforced section. The cracking moment is a function of the section modulus of the uncracked cross section and the modulus of rupture of the concrete. For normal weight concrete the value of this modulus is:

$$f_r = 7.5\sqrt{f_c'}\ \text{(psi)}$$

In order to convert this ultimate stress to working stress the modulus of rupture will be multiplied by a strength reduction factor of 0.80 and divided by a safety factor of 2. This results in an allowable bending stress of:

$$f_b = 3.0\sqrt{f_c'}\ \text{(psi)} \qquad \text{(B-2)}$$

This stress agrees with that presented in paragraph 18.4.1(b) of the same code.

The shearing action of the force on the shear plane *wx* is considered to be one of pure shear rather than diagonal tension. The ACI Code does not establish an allowable stress for pure shear. The Code does establish, however, in paragraphs B.3.1(b) and B.7.4.1, the following allowable stress for shear when the section resists shear in combination with flexure.

$$v_c = 1.1\sqrt{f_c'}\ \text{(psi)} \qquad \text{(B-3)}$$

It is the opinion of the author that the three preceding evaluations provide a method whereby the shear key can be reasonably and conservatively designed. These evaluations are summarized in Table B-1, in which values given are for 3000 psi concrete. This strength concrete was chosen because of its frequent use in foundations and foundation walls.

TABLE B-1. Allowable Stresses for Shear Key Calculation

Type of Stress	General Formula	Allowable Stress[a] (psi)
Bearing f_{brg}	$0.3 f_c'$	900
Flexure f_b	$3.0\sqrt{f_c'}$	164
Shear v_c	$1.1\sqrt{f_c'}$	60

[a] Stresses are based on 3000 psi concrete.

Summary of Stresses

The induced and allowable stresses for bearing, flexure and shear can be equated in terms of the external force *P*. This will result in a correlation between the allowable transfer force and the size of the shear key. This correlation is shown in Table B-2, in which all work is based on one linear foot of wall.

TABLE B-2. Calculations for Shear Key Transfer Force

Type of Stress	Formula	Substitution	Transfer Force P^a
Bearing	$f = \frac{P}{A}$	$900 = \frac{P}{12\ h}$	$10{,}800\ h$
Flexure	$f = \frac{M}{S}$	$164 = \frac{P\ h/2}{12\ b^2/6}$	$\frac{656\ b^2}{h}$
Shear	$f = \frac{P}{A}$	$60 = \frac{P}{12\ b}$	$720\ b$

[a]Transfer force is based on 3000 psi concrete.

B-4. RECOMMENDED SHEAR KEY DIMENSIONS

By equating the values found in Table B-2 for bearing and flexure to that of shear it can be shown that shear will govern in all cases when:

$$15.0\ h > b > 1.1\ h$$

When width and height are kept within these limits, the shear key need only be designed for shear. This is a very desirable simplification of the work.

Shear keys are usually made of finished lumber, such as 2×4^s, 3×6^s, etc. Calculations will recognize that the actual size is one-half inch less than the nominal size.

B-5. RECOMMENDED TRANSFER FORCE

The author has observed the spauling of concrete at the edges of a large percentage of shear keys. For this reason, it is recommended that the width of the shear key be designed as if it were one inch smaller than the actual width specified for construction. The transfer force, based on shear, will be computed from Formula (B-4) and recorded in Table B-3.

TABLE B-3. Allowable Transfer Force on Concrete Shear Keys

Nominal Size Depth × Width	Transfer Force P^a #/ft of Wall
2 × 4	1800 #
3 × 6	3240 #
4 × 8	4680 #
4 × 10	6120 #
6 × 12	7560 #

[a]Transfer forces are based on Formula (B-4), using 3000 psi concrete.

$$P = 720\,(b - 1) \tag{B-4}$$

It may be of interest to note that the shear key at the base of a free standing cantilever retaining wall will be larger than the corresponding key at the base of a basement wall. This is because the key at the retaining wall must transfer all of the earth pressure, whereas the key at the basement wall must transfer only its proportionate share of the pressure.

APPENDIX C

Pressure Distribution Within a Soil Mass

C-1. GENERAL OBSERVATIONS

There are times when it is necessary to determine the vertical pressure, p_v, induced at some specified point within a soil mass due to the action of an applied load. This load will generally be applied to the soil by a foundation whose contact surface will take one of the following forms:

1. A concentrated load, Q, in pounds or kips: It is recognized that there is no such thing as a truly concentrated load. The effect produced by a small area of contact, however, approximates that of a concentrated load and may be used without incurring undue inaccuracies.
2. A uniformly distributed circular load, q, in psf or ksf: Circular loads are generally the result of storage tanks, silos and other similar structures.
3. A rectangular load, q, in psf or ksf: This is the form invariably found in buildings. All spread footings are of this form, and may be square, rectangular, or trapezoidal, as in the case of combined footings.

Bouzzinesq, a French mathematician of the nineteenth century, developed a series of equations which dealt with the distribution of pressure within a solid. For his analysis he assumed that the solid was homogeneous throughout its mass, that it was isotropic (uniform in all directions) and that it was of great extent in all directions. Subsequent to that analysis an engineer by the name of Westergaard published, in 1938, a similar analysis directed specifically to the distribution of pressure within a soil mass. The Westergaard analysis was based on the assumption that the soil consisted of a series of very thin alternating layers of elastic and inelastic material.

Soils are rarely homogeneous, nor are they isotropic. For this reason the

Westergaard analysis is probably a better approximation of real soil conditions. The Bouzzinesq analysis, however, is more frequently used than the Westergaard analysis because it is somewhat easier to work with and is inherently more conservative.

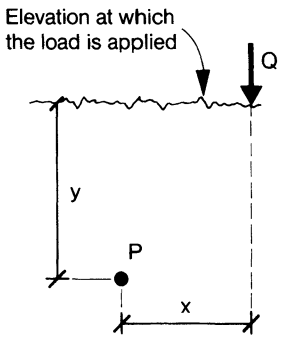
The procedures that follow provide for the determination of the vertical pressure induced at any point within the soil mass. These procedures are based on the assumption that the soil mass extends homogeneously in all directions for a considerable distance beyond the points of interest. This is, obviously, an invalid assumption. Whether these procedures can be used to give reasonably valid approximations is the responsibility of the architect or engineer who must interpret them. It must again be noted that the safe and cost effective design of any kind of foundation demands the proper mixture of technical training, experience, judgment, and intuition.

C-2. PRESSURE INDUCED AT ANY POINT BY A CONCENTRATED LOAD

It is recognized that the load transferred from a foundation to the soil cannot actually be a concentrated load. There must be an area of contact. There are certainly occasions, however, when this approximation can be used without inducing unacceptable error. The decision to use, or not to use this approximation is that of the designer. The parameters by which this approximation can be used is given in Figure C-1.

The Bouzzinesq equation, as given in Figure C-1, may be used to compute the intensity of vertical pressure induced at any point within the soil mass by the action of a concentrated load. In this equation—

Q = the concentrated load (pounds)
p_v = the vertical pressure induced at point P (psf)
C_1 = an influence coefficient, values as given in Table C-1
x and y = distances, as shown in Figure C-1 (feet)



Bouzzinesq equation:

$$p_v = \frac{3Q}{2\pi y^2 \,[1 + (x/y)^2]^{5/2}} = C_1 \frac{Q}{y^2}$$

FIGURE C-1. Pressure induced by a concentrated load.

TABLE C-1. Influence Coefficient C_1

x/y	C_1	x/y	C_1
0	0.477	0.7	0.176
0.1	0.466	0.8	0.139
0.2	0.433	0.9	0.108
0.3	0.385	1.0	0.084
0.4	0.329	1.1	0.066
0.5	0.273	1.2	0.051
0.6	0.221	1.3	0.040

C-3. PRESSURE INDUCED AT ANY POINT BY A CIRCULAR LOAD

The concept of vertical pressure induced at any point in a soil mass by a uniformly distributed circular load was previously introduced in Article 5-5, in which the distribution of pressure was described as a pressure bulb. A bulb relating to a circular footing was illustrated in Figure 5-3. The pressure produced at any point within the soil mass can be read directly from this bulb as a decimal percentage of the contact pressure immediately beneath the footing. To properly use the bulb it is first necessary to determine the coordinates (x) and (y) of the point in question as a multiple of the footing diameter. Refer to Figure C-2.

Although technically inaccurate, this method can usually be applied to square areas whose side is equal to the diameter of the circular load.

C-4. PRESSURE INDUCED AT A CORNER BY A RECTANGULAR LOAD

This procedure may be used to determine the intensity of pressure at any depth directly beneath the four corners of a loaded area. This procedure is applicable

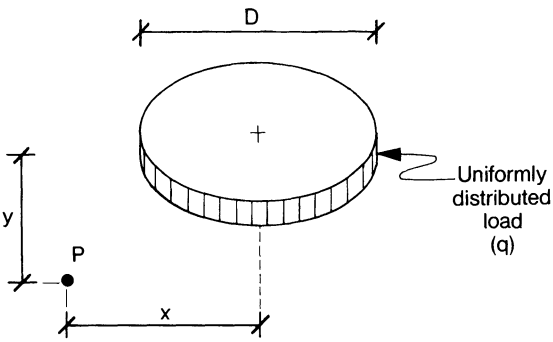


FIGURE C-2. Pressure induced by a circular load. Coordinates x and y must be expressed in terms of the diameter.

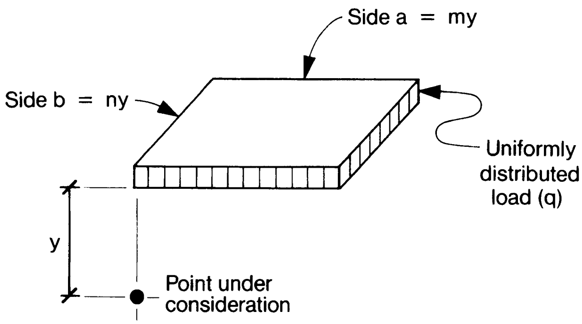


FIGURE C-3. Pressure induced by a rectangular load.

for use with square, rectangular or continuous footings. The parameters required are given in Figure C-3.

The intensity of pressure induced at point P is found as follows:

$$p_v = C_3 q$$

In which:

q = the uniformly distributed load (psf)
p_v = the intensity of vertical pressure (psf)
C_3 = an influence coefficient, given in Figure C-4

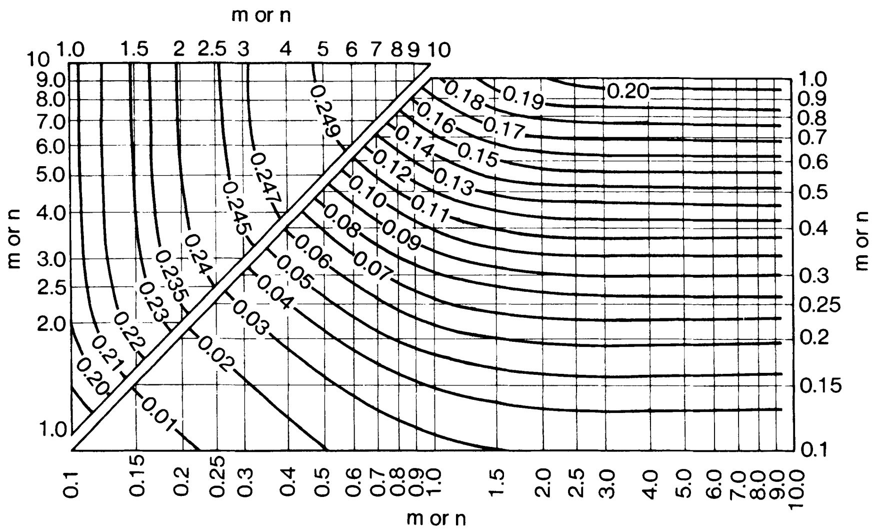


FIGURE C-4. Plot of the influence coefficient C_3. [Ref. 19]

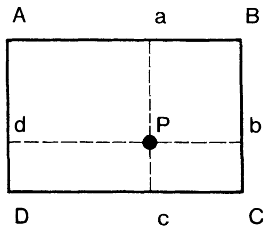


FIGURE C-5. Pressure induced by a rectangular load at an interior point.

C-5. PRESSURE INDUCED AT ANY POINT BY A RECTANGULAR LOAD

The procedure used herein is a modification of the one introduced in Article C-4, in which the pressure was determined directly beneath the corner of the footing. A modification of this procedure permits determination of pressure at any point within the soil mass, either inside of or beyond the extremities of the footing. In order to use this modification the original footing area must be divided into smaller areas specifically selected so that the corners of these areas coincide with the point for which the pressure is required. This procedure is illustrated in Figures C-5 and C-6, in which:

$$p_v = C_3\, q$$

In Figure C-5, the pressure at point P is equal to the additive effect of the following areas:

$$AaPd + aBbP + bCcP + dPcD$$

Note that points A, B, C, and D are the corners of the actual footing. Points a, b, c, and d are the corners of the four pretend footings.

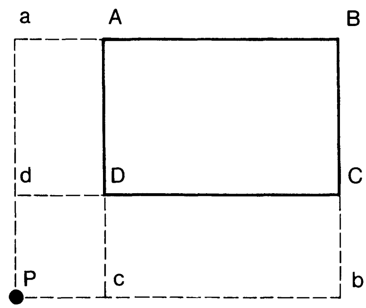


FIGURE C-6. Pressure induced by a rectangular load at an exterior point.

In Figure C-6 the pressure at point P is equal to the additive effect of the following areas:

$$aBbP - aAcP - dCbP + dDcP$$

Note that the effect of area $dDcP$ was added once, subtracted twice, and finally added again for a net effect of zero.

C-6. SAMPLE PROBLEMS

Example C-1

> Required: To determine the vertical pressure induced by a concentrated load of 400 kips at a point 10 feet away from the point and 20 feet below it.

Refer to Figure C-1: Calculate $\frac{x}{y} = \frac{10}{20} = 0.5$

From Table C-1, the coefficient $C_1 = 0.273$

Therefore, the vertical pressure $p_v = 0.273 \frac{400{,}000}{20^2} = 273$ psf

Example C-2

> Required: To determine the vertical pressure induced by a circular load of 6 ksf at a point 8 feet away from the center of the circle and 12 feet below it. The circular area has a diameter of 10 feet.

Refer to Figure 5-3. Calculate

$$x = \left[\frac{8}{10}\right] \mathrm{B} = 0.8\mathrm{B}$$

and

$$y = \left[\frac{12}{10}\right] \mathrm{B} = 1.2\mathrm{B}$$

Using these coordinates, read 0.11q. Therefore, the vertical pressure at the specified point is:

$$p_v = 0.11 \times 6000 = 660 \text{ psf}$$

Example C-3

Required: To determine the vertical pressure induced by a rectangular load of 8 ksf at a point 20 feet below either of the four corners. The rectangular area is 14 feet by 30 feet.

Refer to Figure C-3. Calculate

$$m = \frac{30}{20} = 1.5$$

and

$$n = \frac{14}{20} = 0.7$$

From Figure C-5, the coefficient $C_3 = 0.165$

Therefore, the vertical pressure $p_v = 0.165 \times 8{,}000 = 1{,}320$ psf

Example C-4

Required: To determine the vertical pressure induced 20 feet below a point located 5 feet to the right of center and 5 feet below center of the rectangular area described in Example C-3.

Refer to Figure C-5 for the way in which the large area must be divided into four smaller areas. Compute values for (m) and (n) in accordance with Figure C-3, then obtain the numerical value of the coefficient C_3 from Figure C-4. Computations leading to p_v are shown in Table C-2.

TABLE C-2. Determination of P_v

Area	m	n	C_3	P_v
AaPd	20/20 = 1.0	12/20 = 0.60	0.136	1,088
aBbP	10/20 = 0.5	12/20 = 0.60	0.095	760
bCcP	20/20 = 1.0	2/20 = 0.10	0.029	232
dPcD	10/20 = 0.5	2/20 = 0.10	0.020	160
				2,240 psf

APPENDIX D

Slab on Ground—Nonexpansive Soil

D-1. INTRODUCTION

This appendix deals with slabs cast directly on ground that is not only suitable for bearing but that is also free and clear of fine grained soils exhibiting shrink-swell characteristics. Such soil, commonly referred to as expansive clay, is the subject of Chapter 13.

This appendix, then, deals with ordinary slab on ground in both light commercial and residential construction.

D-2. CONSTRUCTION AS A FUNCTION OF CLIMATE

Houses with basements or crawl spaces are considered normal construction in much of the central and northern parts of the country. This is because of the climate associated with these areas. Winter weather brings freezing weather and any moisture contained within the soil can be frozen to a considerable depth. The depth to which ground moisture can freeze is called the frost line. The maximum anticipated depth to the frost line within the continental United States is given in Figure 7-12.

When water freezes, it expands with an almost irresistible force. Building foundations can sustain catastrophic damage if the ground beneath them freezes. In order to eliminate the possibility of this ever happening, the foundations for all structures must extend below the frost line. It can be seen, therefore, that basements are a natural feature of residences in areas of the country which have a significant frost line.

The temperate climate in the southern part of the country precludes the need to extend foundations below that required for adequate soil bearing pressure. The additional cost of constructing a basement or crawl space solely for the purpose of having one is usually considered to be unwarranted. Light commercial and residences in temperate climates, therefore, are commonly constructed with the first floor slab poured directly on the ground.

D-3. GENERAL DETAILS

Before adopting slab on ground as the right choice for the floor under consideration it must be determined that the soil beneath the slab will provide adequate bearing for the superimposed loads to which it will be subjected. It is the opinion of the author that this soil should have a minimum allowable bearing pressure of at least one tsf in order to be considered adequate for the purpose intended.

Slabs on ground are conventionally built as detailed on Figure D-1. The work consists of a reinforced concrete slab, a vapor barrier, and a stone base, each of which are discussed in subsequent paragraphs.

Slabs on ground must be sized and reinforced to serve the loads adequately to which they will be subjected. Some slabs are lightly loaded, as would be the case in residences and most retail stores. The thickness and reinforcing of such slabs is usually determined empirically. Other slabs may be very heavily loaded, as would be the case in a building used for industrial purposes. The thickness, strength, and reinforcing of all such slabs must be determined by the designer. Some slabs, because of poor soil or close proximity of columns, must be designed as mat foundations. Other slabs may be post-tensioned to add rigidity to the slab and to control shrinkage and temperature cracks.

Lightly Loaded Slabs

Lightly loaded slabs on ground are usually designed empirically, with thickness and reinforcing determined by the experience of past performance. The guidelines

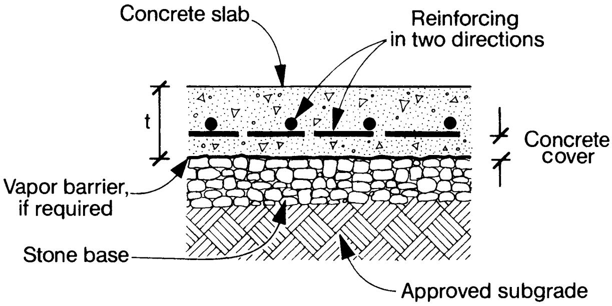


FIGURE D-1. Typical detail of a slab on ground. Concrete cover over reinforcing to be 1½″ for slabs up to 5″ thick, 2″ for slabs over.

TABLE D-1. Empirical Design for Lightly Loaded Slabs on Ground

Minimum Thickness (in.)	Minimum Ultimate Strength (psi)	Reinforcing Each Way[a]	Maximum Load (psf)
4	2,500	# 3 @ 12″	100
5	2,500	# 4 @ 15″	200
6	3,000	# 5 @ 18″	400
8	3,000	# 6 @ 18″	600

[a]Refer to Figure D-1 for location.
[b]Table based on $A_s \geq 0.0018\ A_g$ ACI 7.12.2.1(b) and $S \leq 3t$ or 18″ ACI 7.6.5

given in Table D-1 are based on empirical design, and should be considered as minimum guidelines. These guidelines are based on the assumption that the superimposed load is essentially uniformly distributed and are invalid for conditions where the slab must support heavy concentrated loads. The designer must use his experience and judgment to determine whether this table is applicable to his work.

A lightly loaded, lightly reinforced slab is shown in Figure D-2. Note that the reinforcement in this particular example has not been tied at each intersection even though this should be standard for such widely spaced bars. There is a lack of rigidity in this reinforcing and it can be expected to shift somewhat during concreting.

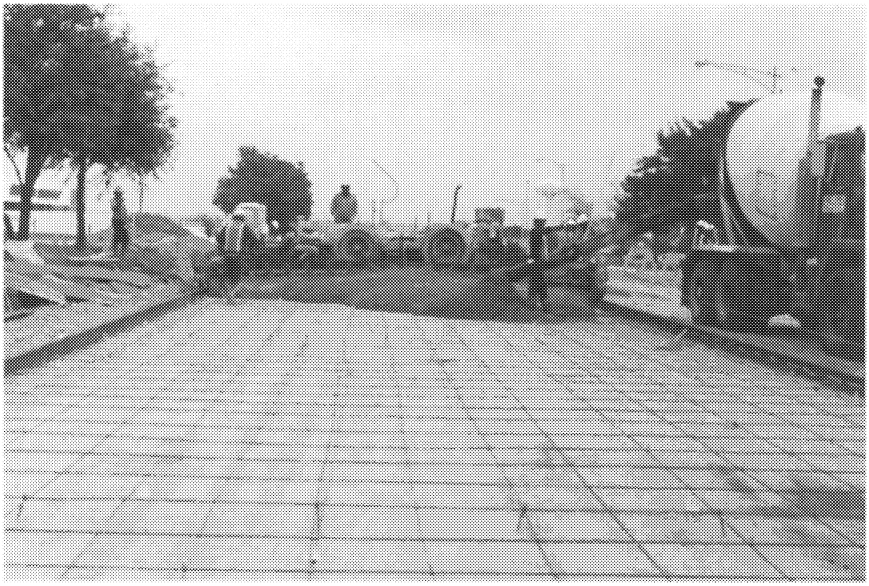


FIGURE D-2. A lightly loaded, lightly reinforced slab on ground.

FIGURE D-3. A heavily loaded, heavily reinforced slab on ground.

Heavily Loaded Slabs

Heavily loaded slabs, particularly those supporting concentrated loads, must be individually designed, taking into consideration the magnitude of the load and the way in which the load is applied to the slab. A heavily loaded, heavily reinforced slab is shown in Figure D-3. Note the use of relatively large diameter bars and close spacing. Note also that reinforcing is placed at the top and bottom of the slab in both directions.

Mat Foundations

Mat foundations are heavily reinforced slabs extending over an extensive area and functioning as a large combined footing. This type of foundation is primarily used in areas of low allowable soil bearing pressure. Column loads, therefore, must be distributed over large floor areas in order to reduce the unit load imposed on the soil. Mat foundations could also be required in areas of closely spaced columns.

The construction of the mat foundation is similar to that shown for the slab on ground in Figure D-1, the difference being that the concrete will be appreciably thicker and much more heavily reinforced.

The design of the mat is analogous to the design of a flat plate, except that it is inverted. It should be remembered that in the analysis of flat plates the occurrence of punching shear around the periphery of the column is a primary design concern and often times is the factor that determines the thickness of the mat.

Mat foundations shall be designed in accordance with the requirements of the ACI Building Code, by engineers familiar with that type of construction.

Prestressed Slabs

Some years ago the Federal Housing Administration authorized a technical study to establish design criteria for residential slabs on ground. The result of this study was to establish four basic slab types based on functional needs and method of construction. These types were identified as types I, II, III, and IV.

Slabs in types I and II are lightly loaded residential slabs constructed on reasonably stable soil. These slabs may be post-tensioned to add rigidity to the slab and to control the occurrence of shrinkage and temperature cracks. The basic design philosophy for these slabs is to provide a residual compression stress at the center of the slab that will be sufficient in magnitude to overcome any incidence of cracking. The tensile force necessary to produce this residual stress must also be large enough to overcome the effect of subgrade friction, elastic shortening and the effects of drying shrinkage. The engineer has the responsibility of determining the required amount of residual compression stress.

As an example of the use of the design process let it be required to provide a residual compression stress of 75 psi in a 5″-thick slab on ground having a length of 120 feet. Computations are based on one foot of width of slab.

Due to symmetry the frictional force is computed on one-half of the slab length. The coefficient of friction has been established by code as 0.5.

Frictional force = weight × effective length × coefficient of friction

$$F = 62.5 \times \frac{120}{2} \times 0.5 = 1{,}875 \text{ \#/ft}$$

$$\text{Residual force required} = 75 \times 5 \times 12 = 4{,}500 \text{ \#/ft}$$

$$\text{Total force required} = 1875 + 4500 = 6{,}375 \text{ \#/ft}$$

In Example 11-9, it was determined that a ½$^{\phi}$ post-tensioned low relaxation tendon having an area of 0.144 square inches and an ultimate strength of 250 ksi would produce a long term residual tensile force of 21.4 kips. The required tendon spacing, therefore, is:

$$\text{Spacing} = \frac{21400}{6375} = 3.35 \text{ feet, say } 3'\text{–}4'' \text{ on centers}$$

Type III and type IV slabs on ground require more comprehensive engineering because of use, loading and method of construction. For information relative to these slabs the reader is directed to a booklet published by the Post-Tensioning Institute entitled Design and Construction of Post-Tensioned Slabs-on-Ground.

D-4. REINFORCING STEEL

General

All reinforcing shall be new billet steel deformed reinforcing bars, conforming to ASTM—A 615, Grade 60. Bars shall be laid in two contacting layers placed at ninety degrees, so as to form a mat or a mesh. Bars shall be wired with galvanized tie wire at alternate intersections in both directions. The mat shall be supported at the proper level on pieces of brick, block or some other kind of precast unit. These units shall be placed no more than four foot centers in each direction. Particular care shall be made to maintain the mat at its proper level before and during the pour.

Splices in Reinforcement

It is customary in many projects to ship reinforcing in stock lengths when such reinforcing is to be used for slabs on ground. Stock lengths, as shipped to the job, are as shown in Figure D-4. Stock lengths are usually around twenty to thirty feet. Reinforcing, therefore, can not extend from one end of the slab to the other end in a single, full length piece, but must be spliced.

It is recommended that splices in reinforcement should be arranged so that no more than every third bar is spliced at one particular point. Splices in adjacent bars should be offset by at least two feet. This arrangement of splices is illustrated in Figure D-5.

When individual bars are spliced, they shall be placed in contact, wired together, and lapped in accordance with the recommendations given in Table D-2. These laps are recommended only for shrinkage and temperature reinforcing in non-working slab on ground situations. If this reinforcement is subject to tension or compression then the laps should be increased to those given in items (4) or (5) in Table E-1.

Wire Mesh Alternate

Welded wire fabric, ASTM-A185 and ASTM-A497 (sometimes referred to as wire mesh) could be used, according to code, as reinforcement in the more lightly reinforced slabs. It has been the author's experience, however, that except for

FIGURE D-4. Reinforcing in stock lengths as shipped to the job site.

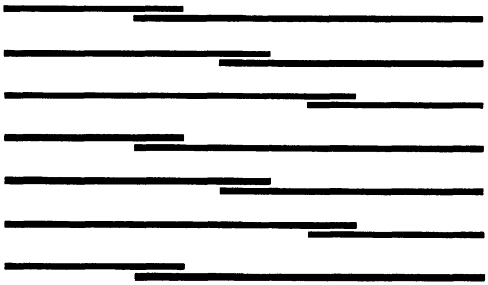


FIGURE D-5. Recommended arrangement for the splicing of reinforcing in slabs on ground.

TABLE D-2. Minimum Lap Length for Spliced Bars in Slabs on Ground

Bar Size	Lap Length (in.)
#3	12
#4	15
#5	18
#6	21

Note: Splice no more than one-third of the bars at any point. Refer to Figure D-3.

the very largest of wires, wire mesh is too flimsy to survive the rigors of construction inherent with working directly on earth or stone subgrade. Mesh is very difficult to keep at the specified height within the slab, and the individual wires are easily bent out of shape. Reinforcing bars are the only reasonable way in which to reinforce the slab because of their relative stiffness and the ease with which they can be maintained in proper position before and during the pour. Note, however, that many contractors have a strong preference for mesh reinforcement due to its relative economy and ease of initial placement, and they may make a hard sell to have a mesh substitution approved by the designer. The designer should not be talked into this substitution without careful consideration.

D-5. STONE BASE

General

After it has been determined that the soil beneath the slab will perform satisfactorily as subgrade, the subgrade should be leveled. Soft spots should be removed and filled. Hard spots should be cut out to a depth of six inches and filled. If required, borrow fill should conform to the requirements of Article 12-2.

The stone base is now installed. The purpose of this base is twofold: first, to provide frictional bonding between the slab and earth, and second, to provide a place for the collection and disposal of any water or water vapor that might infiltrate the area. It is very important, therefore, that the base be free of any appreciable amount of rock dust, screenings, fines or any other material that would adversely affect permeability.

Material

Broken stone (crushed rock) well graded from one and one-half inches to pea gravel size is recommended as a very satisfactory material to be used for the stone base. Other materials such as USCS Groups GW, GP, GM, and SW are equally acceptable. It will generally be found, however, that the broken stone base will be more economical.

The recommended thickness of the stone base is as follows:

1. Four-inch base—for slabs up to 6 inches in thickness
2. Six-inch base—for slabs over 6 inches in thickness

Installation

Many contractors prefer to install the stone base, at least in part, before proceding with construction of the building proper. There are three real advantages in doing this:

1. The base material can be trucked directly to the place of use.
2. The bulk of the compaction process can be performed with heavy, outdoor machinery.
3. The construction of the building can proceed on a relatively clean, dry working area, rather than in the mud.

In those instances when delivery of the stone base must be delayed, or when additional material is required, the contractor will leave an access through which the base material can be brought. Such an access is illustrated in Figure D-6.

Compaction of the stone base should be accomplished by working large, open areas with heavy machinery equipped with pneumatic tires or steel-wheeled rollers in accordance with the recommendations included in Article 12-4. Smaller areas should be compacted by the use of smaller, hand guided equipment as noted in Article 12-5.

Compaction should result in a well-seated, well-graded stone base, similar to that illustrated in Figure 12-8(c). Because compaction of the stone base can be observed in the field, it is usually not necessary to require field density tests. It is far more important to make sure that the contractor understands what is required in the end result, and to see that the equipment and methods used will achieve that result.

FIGURE D-6. Access as required for the delivery of stone base for use under slab on ground. [Ref. 7]

Vapor Barrier

A 6 mil polyethylene vapor barrier can be installed directly over the stone base if one is required in order to prevent the migration of water vapor into a finished space. In this instance the stone base should be top coated with a layer of coarse sand. This will serve to protect the vapor barrier from any sharp corners or edges in the stones.

D-6. FINISHING

The easiest way to pour a slab on ground is when the slab is accessible to the concrete truck. The concrete can then be delivered directly to the point of deposit by the use of a chute attached to the truck hopper, as illustrated in Figure D-7. Concrete collects in the hopper and flows sluggishly down the chute by gravity. The concrete is frequently pushed along the chute by workmen using shovels or rakes. The concrete is then manually brought to rough grade.

Finishing of the concrete can be done manually or mechanically. When performed manually the surface is first floated, usually with wood floats that are worked across the surface. The surface is then checked for level and any out of level is corrected. Final finishing is usually done with steel trowel. Mechanical finishing requires major equipment and site access. For these reasons it is usually reserved for outdoor construction, particularly highways. A typical mechanical operation is illustrated in Figure D-8.

FIGURE D-7. Concrete deposited to slab on ground by chute.

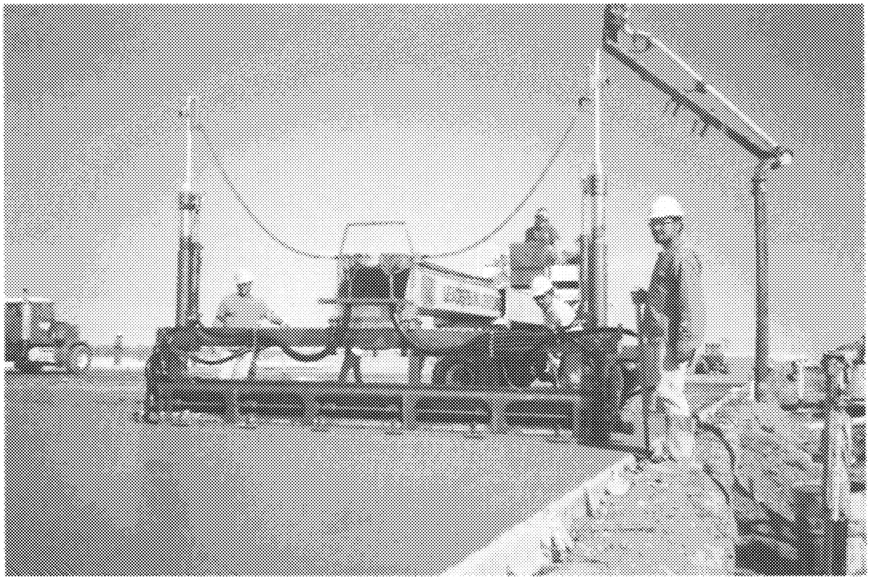


FIGURE D-8. Mechanical finishing of a highway slab.

D-7. GROUND WATER

The foregoing paragraphs assume that the occurrence of ground water need not be considered. If ground water does occur, then all details regarding the slab, waterproofing, and water control will change dramatically. The work must then be redesigned by engineers knowledgeable in the problems relating to hydrostatic pressure and water control.

APPENDIX E

Dowels for Load Transfer into Footings

E-1. GENERAL CONSIDERATIONS

Purpose of Dowel

All walls and columns contain vertical reinforcing bars designed to carry their proportionate share of the total load carried by the element. This load may be tension or compression. As discussed in Article 7-7 the purpose of a dowel is to transfer the load carried by these reinforcing bars into the footing. The load is first transferred from the bars into the dowels, and then from the dowels into the footing.

Dowels are usually designed to be equal in size and number to the bars being doweled, and they are usually placed so as to be in contact with the bars being doweled. A typical arrangement of the placement of dowels between a footing and a column has been shown in Figure 7-9.

Load Transfer Through Dowels

Reinforced concrete works because of the interaction developed between the reinforcing and the concrete. This interaction is a function of the physical interlocking of the two materials as produced by the deformations on the reinforcing steel, and of the adhesion developed between the two materials on their surface of contact. It may be of interest to note that the development of adhesion is somewhat improved when the surface of the reinforcing steel has oxidized to form a very thin, tight coating of rust.

Transfer of load from the main reinforcing steel to the dowel is accomplished by providing a length of contact called the lap splice length. Transfer of load from the dowel to the footing is accomplished by extending the reinforcement into the footing for a certain specified length called the development length. The concept of lap splice and development lengths is based on the attainable average bond stress developed on the surface of contact between the reinforcing and the concrete.

Typical Dowel Requirements

A typical detail of a reinforcing dowel is illustrated in Figure E-1. The vertical length of the dowel is divided into two parts, one above and one below the top of the footing. The length required in each part is based on the following requirements:

1. Each dowel must extend into the wall or column the distance required to fully develop the calculated axial tension or compression in the vertical reinforcing of the wall or column. This distance is referred to as the lap splice length because it is through this length that the dowel and the vertical bar are lapped for the purpose of load transfer.
2. Each dowel must extend into the footing the distance required to fully develop the calculated axial tension or compression in the dowel by means of bond. This distance is referred to as the development length, and is symbolized by l_d for straight bars, and by l_{dh} for hooked bars.

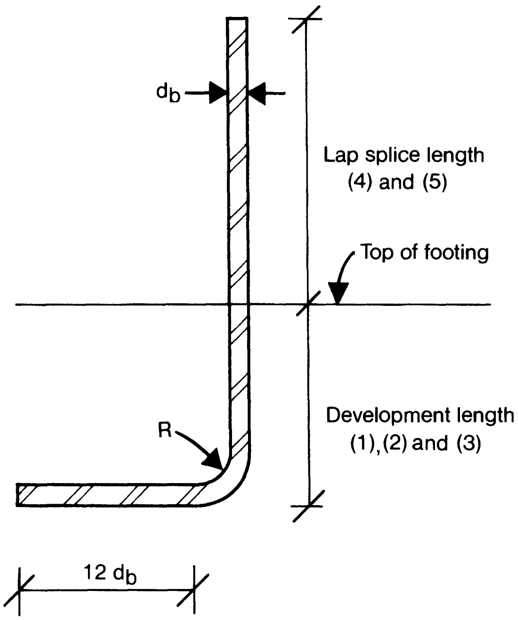


FIGURE E-1. Typical dowel detail.

E-2. LENGTH REQUIREMENTS

Selected requirements relative to development and lap splice lengths, as established by ACI 318-83, are given in the following paragraphs. This information has been abridged to include only those requirements relative to the following:

1. The dowels are used solely for the purpose of transferring vertical load from wall or column reinforcing into the footing.
2. Unit weight of concrete is 150 pcf, ultimate compression strength is 3000 psi.
3. Yield stress of reinforcing steel is 60,000 psi.

For additional requirements relative to the use of dowels refer to the Building Code Requirements for Reinforced Concrete, ACI 318-83 [Ref. 5].

The requirements by which lap splice lengths and development lengths are determined is as follows:

1. Development length l_d for straight bars—Tension—ACI 12-2

   $$l_d \text{ shall equal } l_{db}\text{, but not less than } 12''$$

   For #11 bars and smaller—

   $$l_{db} = 0.04\ A_b f_y \ / \ \sqrt{f'_c}\text{, but not less than } 0.0004\ d_b f_y$$

   Under certain conditions of bar spacing, excess reinforcement or spirals the length l_{db} can be reduced. Refer to ACI 12.2.4.
2. Development length l_{dh} for hooked bars—Tension—ACI 12.5

   $$l_{dh} \text{ shall equal } l_{hb}\text{, but not less than } 8d_b \text{ nor } 6''$$
   $$l_{hb} = 1200\ d_b \ / \ \sqrt{f'_c}$$

   Under certain conditions of bar cover, ties or excess reinforcement the length d_{hb} can be reduced. Refer to ACI 12.5.3.
3. Development length l_d for straight bars—Compression—ACI 12.3

   $$l_d \text{ shall equal } l_{db}\text{, but not less than } 8''$$
   $$l_{db} = 0.02\ d_b f_y / \sqrt{f'_c}\text{, but not less than } 0.0003\ d_b f_y$$

   Under certain conditions of excess reinforcement or spirals the length l_{db} can be reduced. Refer to ACI 12.3.3.
4. Lap splice length—Tension—ACI 12.15
   Assuming a Class B splice, the lap splice length l_d shall be:

   $$l_d = 1.3 \text{ times the development length from (1)}$$

   A Class B splice limits the number of bars being spliced to no more than 50% of the total number of bars at that point and is valid when A_s provided is less than twice A_s required

5. Lap splice length—Compression—ACI 12.16
   The lap splice length l_d shall be equal to the development length l_d from (3), but not less than 0.0005 $d_b f_y$ nor 12″
   Under certain conditions, the length l_d can be reduced. Refer to ACI 12.16.

Lengths based upon the preceding requirements, and as applicable to wall and column dowels in footings, are given in Table E-1.

TABLE E-1. Development and Lap Splice Lengths for Reinforcing Dowels in Footings

Bar Size	(1)	(2)	(3)	(4)	(5)
#3	12″	9″	9″	12″	12″
#4	12	11	11	16	15
#5	15	14	14	20	19
#6	20	17	17	25	23
#7	27	20	20	35	27
#8	35	22	22	45*	30
#9	44*	25	25	57*	34
#10	56*	28	28	73*	39*
#11	69*	31	31	89*	43*

Note: Lengths are based on 3000 psi ultimate strength concrete and 60,000 psi yield strength steel.
(1) Development length—straight bars—tension
(2) Development length—hooked bars—tension
(3) Development length—straight bars—compression
(4) Lap splice length—tension
(5) Lap splice length—compression

E-3. THE USE OF HOOKS

The development length of a dowel in tension can be significantly reduced by the addition of a hook. This can be seen by comparing the lengths given as items (1) and (2). Without hooks the footing may have to be increased to an unreasonable thickness just to accommodate the longer development length. In footings, therefore, hooked dowels are almost universally used for tensile reinforcing in preference to straight bars. Even with hooks it should be noted that the thickness of the footing may be dependent on bar size. The required extension of the bat into the footing is given in item (2). The thickness of the footing would have to be at least 4″ more than the required bar extension.

Hooks have been found to be ineffective in reducing the development length of a dowel in compression. On the other hand, the use of a hook is very desirable because it facilitates the placement and stabilization of the dowel before and during the depositing of footing concrete. For this reason it is recommended that all footing dowels be detailed with hooks.

E-4. SIZE LIMITATIONS TO THE USE OF DOWELS

As noted in Article 7-7 the method of welding the column bars directly onto the dowels is particularly encouraged in heavily reinforced columns or in columns having large bars. There are two reasons for this:

1. In heavily reinforced columns, the height in which the lapped splice occurs is congested because it would contain twice as many bars as the specified reinforcing. The ACI Building Code, in paragraph 7.6.3, requires the clear distance between longitudinal bars in columns to be not less than 1½ d_b nor 1½″. Although the vertical steel and its dowel are placed in contact the clear distance between adjacent sets of bar plus dowel must satisfy the code. This condition can frequently require an increase in the size of the column.
2. It seems reasonable that there should be a limit to the length of a lapped splice. It is suggested by the author that the limit be set at 36 inches. Bars exceeding that limit are marked with an * in items (4) and (5).

E-5. SIZE SUBSTITUTION—COMPRESSION BARS

The dowels used in tension lap splices must be equal in size and number to the vertical bars being doweled. In compression lap splices, however, the ACI Code permits the use of smaller dowels provided that the lap splice length is the larger of the following:

1. The development length of the larger bar
2. The lap splice length of the smaller bar

Development lengths and lap splice lengths for compression bars are given in items (3) and (5). There are times when the development length of the scheduled vertical bar will not fit into the footing without an increase in footing thickness. In this case there may be an advantage in using smaller dowels because of their relatively smaller development length. When smaller dowels are used the total cross-sectional area of the substitute dowels must be equal to that of the original dowels, as given in Table E-2.

TABLE E-2. Schedule of Substitutions for Compression Dowels

Substitute These Bars	With These Bars
1–#11	2–#8
1–#10	1–#8 and 1–#7
1–# 9	1–#7 and 1–#6
1–# 8	2–#6
1–# 7	2–#5

APPENDIX F

Buoyancy

F-1. GENERAL

Buoyancy may be defined as the condition in which a body immersed in a fluid experiences an apparent loss of weight numerically equal to the weight of the volume of fluid displaced.

In the design of buildings and other structures buoyancy is an important consideration in those instances where the water table is above any part of the element under design. Elements affected by high water table include:

1. Basements
2. Storage tanks
3. Underground vaults
4. Footings

Basements, storage tanks and underground vaults are highly susceptible to the problems caused by high water table. The reason for this is that much of the volume of water displacement is displaced by air. These three elements have a much larger volume to weight ratio than does a footing.

Safety Factor

The safety factor against uplift may be defined as follows:

$$SF = \frac{\text{minimum dead load permanently in place}}{\text{buoyant force due to the highest expected water table}}$$

Safety factors against uplift should meet the following minimum standards:

1. $SF = 2.0$ for basements and other parts of buildings subject to severe damage due to uplift
2. $SF = 1.5$ for underground tanks and vaults not structurally connected to the building proper

It is important to understand that only dead load (and dead load which is permanent) can be counted on to resist the forces of uplift.

It is also important to understand that ground water may fluctuate with the change in seasons and that design must be based on the highest water table which can be expected to occur during the anticipated life of the building.

F-2. SAMPLE PROBLEMS

Example F-1

Required: To determine the condition of buoyancy of the underground storage tank shown in Figure F-1, assuming the tank is filled with oil having a density of 50 pcf.

Given: The total weights are as follows:

Tank = 3,600 #, oil = 30,200 # and concrete pad = 23,600 #.

Volume of water displaced by the partially submerged tank:

$$V = \frac{\pi\ 8.0^2}{4} \times 5.0 + \frac{\pi\ 10.0^2}{4} \times 2.0 = 408.4 \text{ CF}$$

$$\text{Buoyancy W} = 408.4 \times 62.4 = 25{,}500\ \#$$

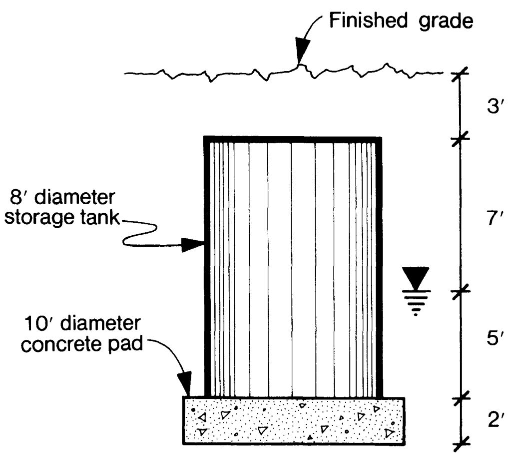


FIGURE F-1. Example F-1—Underground storage tank subject to buoyancy.

With tank full of oil:

$$\text{Total weight} = 3{,}600 + 30{,}200 + 23{,}600 = 57{,}400\ \#$$
$$\text{Safety factor against uplift} = \frac{57400}{25500} = 2.25$$

Example F-2

Required: To determine the condition of buoyancy of the tank illustrated in Example F-1, assuming the tank to be empty.

$$\text{Total weight} = 3{,}600 + 23{,}600 = 27{,}200\ \#$$
$$\text{Safety factor against uplift} = \frac{27200}{25500} = 1.07$$

The tank is very close to lift-off.

Example F-3

Required: To suggest possible solutions for the condition of flotation as determined in Example F-2.

Solution #1: Anchor the tank to the concrete pad and then anchor the pad to sound bearing with prestressed tiedowns. Calculations involving the use of tiedowns will be demonstrated in Example F-3.

Solution #2: Anchor tank to the concrete pad and pour a thicker pad. As an example of this solution assume a five foot thick pad. With the tank empty:

$$\text{Total weight} = 3{,}600 + \frac{\pi\ 10.0^2}{4} \times 5.0 \times 150 = 62{,}500\ \#$$

Volume of water displaced by the partially submerged tank:

$$V = \frac{\pi\ 8.0^2}{4} \times 5.0 + \frac{\pi\ 10.0^2}{4} \times 5.0 = 644\ \text{CF}$$
$$\textit{Buoyancy} = 644 \times 62.4 = 40{,}200$$
$$\text{Safety factor against uplift} = \frac{62500}{40200} = 1.55$$

Since this safety factor is greater than 1.50, the design is satisfactory.

Example F-4.

Required: To determine the effect of ground water on the building indicated in Figure F-2. A detail of the construction of the basement floor is shown in Figure F-3.

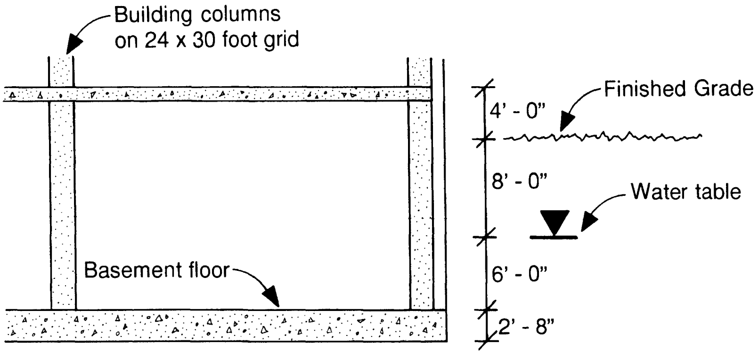


FIGURE F-2. Example F-4—Cross section through building showing general arrangement.

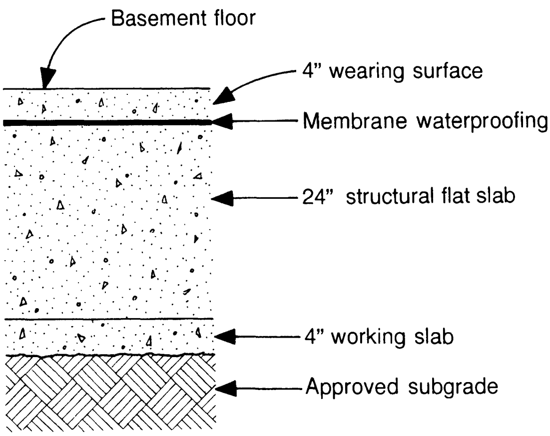


FIGURE F-3. Example F-4—Detail of pressure slab.

On a square foot of floor area:

$$\begin{aligned}\text{Weight of basement slab} &= 2.67 \times 150 = 400 \text{ psf}\\ \text{Buoyancy} &= 8.67 \times 62.4 = 541 \text{ psf}\\ \text{Net uplift} &= 141 \text{ psf}\end{aligned}$$

There are three general procedures by which this net uplift can be overcome:

1. Increase the slab thickness as was done in the previous problem. When a slab is increased to resist water pressure it is frequently referred to as a pressure slab. Note that as the slab thickness is increased the buoyancy is also increased. With a high water table this procedure is usually not practical.

2. Use the dead load of the structure acting through the columns into the pressure slab. The slab in this instance would be designed as an inverted flat plate spanning between columns.
3. Anchor the pressure slab to sound rock with post-tensioned, grouted tendons. The slab in this instance would be designed as an inverted flat plate spanning between the points of anchorage.

Calculations Using Procedure #2:

Assuming a column dead load of 350 kips, the square foot equivalent is:

$$\text{The building dead load contributes } \frac{350000}{24 \times 30} = 486 \text{ psf}$$

$$\text{Safety factor against uplift} = \frac{486 + 400}{541} = 1.64$$

This safety factor is less than 2, which is the recommended minimum for major building construction. In addition to that, there is a problem of construction time. In order for this column to develop the given dead load the building would have to be multistorey. The full impact of the dead load will not be felt until all the floors have been poured. This will take a while, during which time the site must be continuously dewatered. This can be a problem not only in construction but also in scheduling and in costs. The existence of a high water table will be known after the test borings have been completed. When the boring logs are available the architect and engineer should consult with a general contractor experienced in high water table construction. Various solutions can be explored at this time and the course of action determined before the working drawing stage of the project.

Calculations Using Procedure #3:

This procedure involves the installation of prestressed tiedowns to counteract the buoyant effect. This procedure can only be used when the site is underlaid with sound rock at a reasonable depth.

The same general procedure developed in Examples 11-9 and 11-10 will be used to anchor this pressure slab to the underlying rock. In those examples the tiedowns were prestressed tendons having a nominal diameter of one-half inch and an area of 0.144 square inches. The permissible jacking force per tendon was 28.0 kips. This resulted in a long term effective prestress of 21.4 kips.

In this example we will analyse a two strand tendon placed on a grid 10′–0″ on centers. The effective long term tiedown force available to resist uplift will be:

$$\frac{2 \times 21400}{10 \times 10} = 428 \text{ psf}$$

and the resultant safety against uplift will be:

$$\text{Safety factor} = \frac{428 + 400}{541} = 1.53$$

This safety factor is acceptable because it can be considered as a temporary measure occurring only during construction. As each floor is poured the dead load will increase. After all floors have been poured the total dead load available to resist uplift is the additive effects of the tiedowns, the pressure slab and the building. The resultant safety factor will be:

$$\text{Safety factor} = \frac{428 + 400 + 486}{541} = 2.43$$

If the tiedowns plus the weight of the pressure slab had provided sufficient resistance against uplift then the dead load of the building need not be considered.

In order for this tiedown system to work the following must occur:

1. The jacking force must be transferred to the grout pocket.
2. This force must then be transferred from the grout pocket to the bedrock.
3. The tendon must engage sufficient dead weight of rock to fully develop the long term effective prestress in the tendon.

The two strand tendon used in this example must transfer a jacking force of 56.0 kips into the grout. In Example 11-10 the required length of transfer was 5.4 feet, based on a certain ratio of transfer force to tendon perimeter. Since this ratio has not changed the 5.4 feet of pocket length is satisfactory for this part of the transfer.

In the grout to rock transfer a 4″ diameter bore hole will be used. The ratio of transfer force to pocket perimeter is doubled. The previously determined length of 1.5 feet should likewise be doubled. A pocket length of 5′–6″ will again be satisfactory.

The volume of rock engaged by each tendon is conservatively assumed to be a cone whose sides slope one foot horizontal for each two feet vertical. The dead load available to the tendon will then be the weight of this cone of rock plus the weight of any soil above the cone. This is illustrated in Figure F-4. After several trial and errors, (x) is assumed to be 8′–6″ in the calculations which follow.

Note that the weight of the sand layer has been reduced by the buoyancy effect since water will surely permeate this entire layer. The bedrock, however, will not be subject to the buoyancy effect and its weight may be used without reduction.

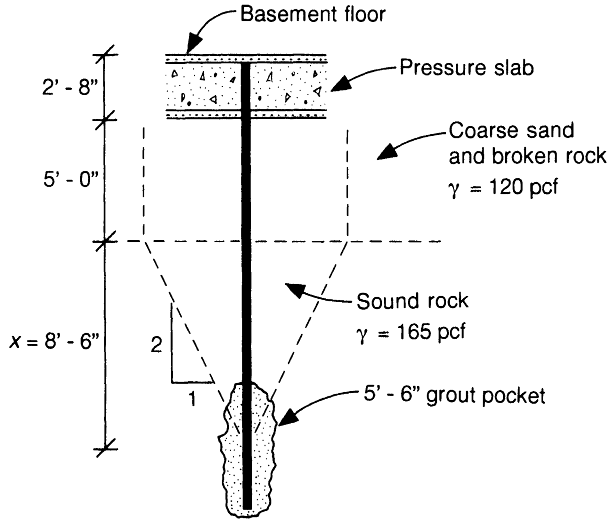


FIGURE F-4. Example F-4—Detail showing volume of bedrock engaged by the tendon.

$$\text{Area at top of cone} = \frac{\pi\ 8.5^2}{4} = 56.7 \text{ sf}$$

$$\text{Weight of soil above rock} = 56.7 \times 5\ (120 - 62.4) = 16{,}300\ \#$$

$$\text{Weight of rock cone} = \left[\frac{1}{3}\right] 56.7 \times 8.5 \times 165 = 26{,}500\ \#$$

$$\text{Total dead load available per tendon} = 42{,}800\ \#$$

The third condition, therefore, is satisfied.

APPENDIX G

The Mathematics of K_a

G-1. INTRODUCTION

This appendix covers the mathematics by which Formulas (9-4) and (9-5) were developed. It is based on the fact that for a given soil the coefficient of active pressure K_a is numerically a function of the angle of rupture (α) as defined in Figure 9-3. This angle in turn is a function of the angle of internal friction. The work regarding these formulas then, involves determining the angle of rupture that will produce the maximum coefficient of active pressure. This coefficient, as introduced in Article 9-5 is repeated herein:

$$K_a = \frac{1 - \text{Tan}\ \phi\ \text{Tan}\ \alpha}{\dfrac{\text{Tan}\ \phi}{\text{Tan}\ \alpha} + 1} \qquad (9\text{-}3)$$

G-2. THE MATHEMATICS OF TAN α

To determine the maximum value of K_a Formula (9-3) must first be differentiated with respect to the tangent of the angle α. In this process note that for a given soil the angle ϕ is constant.

$$\text{Using the derivative form } \frac{\text{du}}{\text{dv}}, \qquad \text{then} \qquad \frac{dK_a}{d\alpha} =$$

$$\frac{\left[\dfrac{\text{Tan}\ \phi}{\text{Tan}\ \alpha} + 1\right][-\text{Tan}\ \phi\ \text{Sec}^2\ \alpha] - [1 - \text{Tan}\ \phi\ \text{Tan}\ \alpha]\left[\dfrac{[-\text{Tan}\ \phi\ \text{Sec}^2\ \alpha]}{\text{Tan}^2\ \alpha}\right]}{\left[\dfrac{\text{Tan}\ \phi}{\text{Tan}\ \alpha} + 1\right]^2}$$

The next step in this process is to set the above equation equal to zero and solve for Tan α. Note that in simplifying:

(a) the term [$-$ Tan ϕ Sec^2 α] in the numerator cancels out
(b) the denominator cross-multiplies out

Therefore: $$\left[\frac{\text{Tan } \phi}{\text{Tan } \alpha} + 1\right] - [1 - \text{Tan } \phi \text{ Tan } \alpha]\left[\frac{1}{\text{Tan}^2 \alpha}\right] = 0$$

Simplifying: $$\left[\frac{\text{Tan } \phi + \text{Tan } \alpha}{\text{Tan } \alpha}\right] - \frac{[1 - \text{Tan } \phi \text{ Tan } \alpha]}{\text{Tan}^2 \alpha} = 0$$

From which: $$[\text{Tan } \phi \text{ Tan } \alpha + \text{Tan}^2 \alpha] - [1 - \text{Tan } \phi \text{ Tan } \alpha] = 0$$

Simplifying: $$\text{Tan}^2 \alpha + 2 \text{ Tan } \phi \text{ Tan } \alpha - 1 = 0$$

And by using the quadratic formula:

$$\text{Tan } \alpha = \frac{-2 \text{ Tan } \phi \pm \sqrt{4 \text{ Tan}^2 \phi + 4}}{2}$$

Simplifying: $$\text{Tan } \alpha = -\text{Tan } \phi \pm \sqrt{\text{Tan}^2 \phi + 1}$$

Substituting the trigonometric identity of $\text{Tan}^2 \phi + 1 = \text{Sec}^2 \phi$, and using the positive sign for the radical:

Then: $$\text{Tan } \alpha = \text{Sec } \phi - \text{Tan } \phi$$

Substituting the following trigonometry identities:

$$\text{Sec } \phi = \frac{1}{\text{Cos } \phi} \quad \text{and} \quad \text{Tan } \phi = \frac{\text{Sin } \phi}{\text{Cos } \phi}$$

Then: $$\text{Tan } \alpha = \frac{1 - \text{Sin } \phi}{\text{Cos } \phi}$$

With the following trigonometric identities:

$$\text{Sin } \phi = \text{Cos } [90° - \phi] \quad \text{and} \quad \text{Cos } \phi = \text{Sin } [90° - \phi]$$

Then: $$\text{Tan } \alpha = \frac{1 - \text{Cos } [90° - \phi]}{\text{Sin } [90° - \phi]}$$

This is of the following form:

$$\text{Tan } \frac{x}{2} = \frac{1 - \cos x}{\sin x} \text{ where } x = 90° - \phi$$

Therefore: $$\text{Tan } \alpha = \text{Tan}\left[\frac{90° - \phi}{2}\right]$$

From which: $$\text{Tan } \alpha = \text{Tan}\left[45° - \frac{\phi}{2}\right]$$

This is the value of Tan α that will produce the maximum value of K_a. Therefore, the plane of rupture is defined by the following:

$$\alpha = 45° - \frac{\phi}{2} \tag{9-4}$$

G-3. THE MATHEMATICS OF K_a

The tangent of the angle of rupture which will produce the maximum value of the coefficient of active pressure can be identified by either of the following expressions:

$$\text{Tan } \alpha = \text{Sec } \phi - \text{Tan } \phi = \frac{1 - \text{Sin } \phi}{\text{Cos } \phi} = \text{Tan}\left[45° - \frac{\phi}{2}\right]$$

In order to find the maximum value of the coefficient the first of the three expressions will be substituted into Formula (9-3):

$$K_a = \frac{1 - \text{Tan } \phi\,[\text{Sec } \phi - \text{Tan } \phi]}{\dfrac{\text{Tan } \phi + [\text{Sec } \phi - \text{Tan } \phi]}{[\text{Sec } \phi - \text{Tan } \phi]}}$$

Simplifying: $$K_a = \frac{1 - \text{Tan } \phi \text{ Sec } \phi + \text{Tan}^2 \phi}{\dfrac{\text{Sec } \phi}{[\text{Sec } \phi - \text{Tan } \phi]}}$$

Using the trigonometric identify: $1 + \text{Tan}^2 \phi = \text{Sec}^2 \phi$

Then: $$K_a = \frac{\text{Sec}^2 \phi - \text{Tan } \phi \text{ Sec } \phi}{\dfrac{\text{Sec } \phi}{[\text{Sec } \phi - \text{Tan } \phi]}}$$

Simplifying: $$K_a = [\text{Sec } \phi - \text{Tan } \phi]^2$$

Therefore: $$K_a = \text{Tan}^2 \alpha$$

Finally: $$K_a = \text{Tan}^2\left[45° - \frac{\phi}{2}\right] \tag{9-5}$$

APPENDIX H

The Mathematics of Mohr's Circle

H-1. PROOF THAT THE LOCUS OF POINTS IN FIGURE 4-5 IS A CIRCLE

That the locus of points in Figure 4-5 is a circle may be proved by examining the information given in Figure 4-6.

If we assume that the locus of points is a circle, then the Pythagorean theorem applies, with which:

$$\text{Horizontal side } H^2 + \text{Vertical side } V^2 = \text{Radius R}^2$$

$$\begin{aligned} H &= p_3 + 0.5\ (p_1 - p_3) - p \\ &= p_3 + 0.5\ (p_1 - p_3) - 0.5\ (p_1 + p_3) - 0.5\ (p_1 - p_3) \cos 2i \\ &= -\ 0.5\ (p_1 - p_3) \cos 2\mathrm{i} \end{aligned}$$

$$V = 0.5\ (p_1 - p_3) \sin 2\mathrm{i}$$

$$\begin{aligned} H^2 + V^2 &= 0.25\ (p_1 - p_3)^2 \cos^2 2i + 0.25\ (p_1 - p_3)^2 \sin^2 2i \\ &= 0.25\ (p_1 - p_3)^2\ [\cos^2 2i + \sin^2 2i] \\ &= 0.25\ (p_1 - p_3)^2 \end{aligned}$$

Therefore R = $0.5\ (p_1 - p_3)$ QED.

H-2. PROOF THAT THE CENTRAL ANGLE *BCD* IN FIGURE 4-6 EQUALS $2i$

In the triangle *CAD* side *CA* equals side *CD*; therefore, the triangle is isosceles and the base angles are equal.

$$\text{angle } CAD = CDA = i$$

All three angles must add to 180°; therefore:

$$\text{Angle } ACD = 180° - i - i = 180° - 2i$$

The sum of the two angles around point *C* must equal 180°; therefore:

The central angle $BCD = 180° - ACD = 180° - (180° - 2i) = 2i$ *QED*.

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