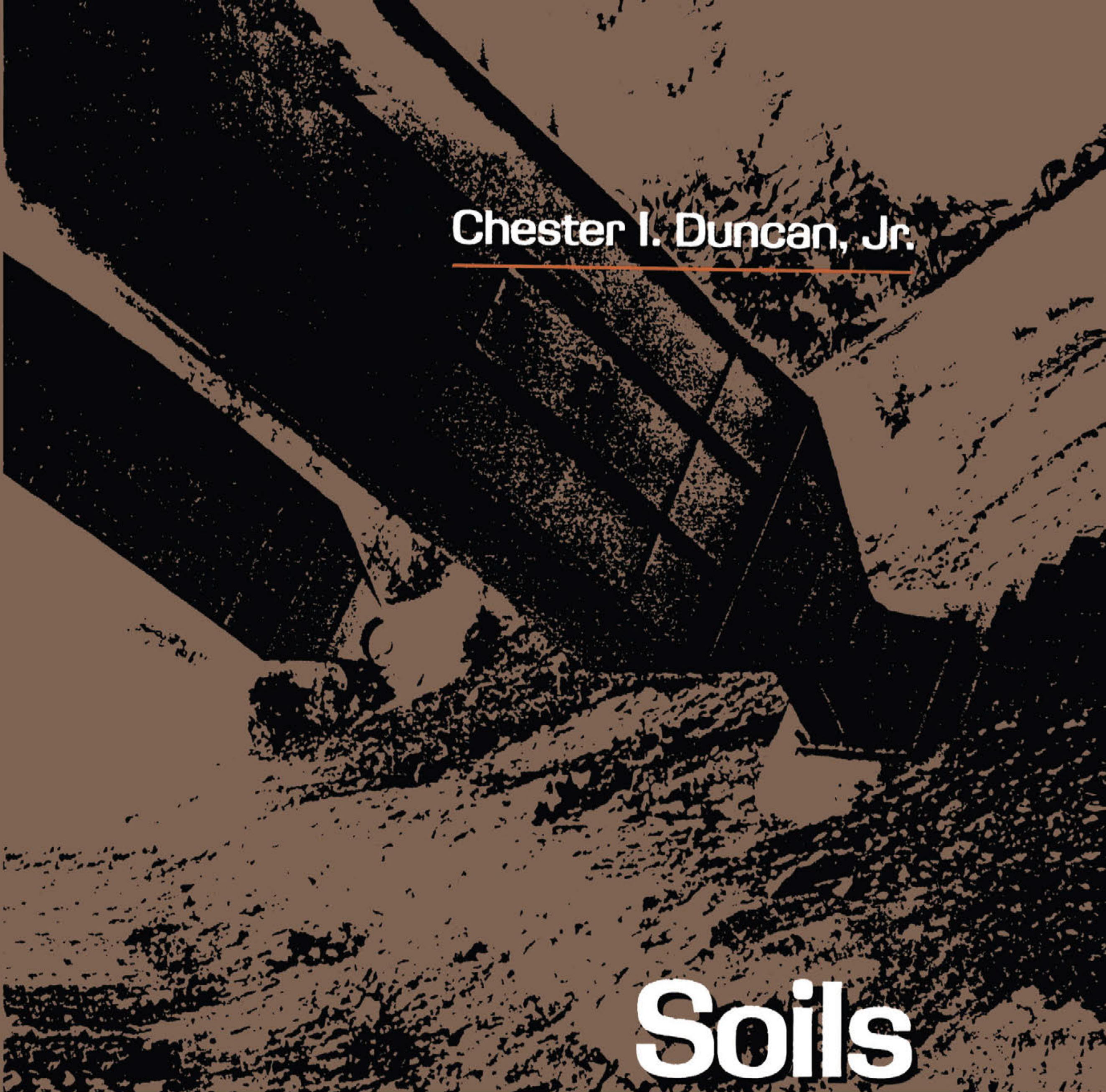

Chester I. Duncan, Jr.

Soils and Foundations

For Architects and Engineers

STRUCTURAL Engineering series

Soils and Foundations
for
Architects and Engineers

Soils and Foundations for Architects and Engineers

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

STRUCTURAL Engineering series

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Sherry Leigh Penley
The University of Texas at Arlington

Contents

Preface

The purpose of this book is twofold:

1. To serve as a textbook for architectural and engineering students in undergraduate and graduate level courses, and
2. To serve as a reference for architectural and engineering practitioners.

The student is interested in the basic theory of soil mechanics and foundations, and in a generalized overview of the application of that theory in practice. The practitioner, on the other hand has learned the basics, and is now interested in the actual application of theory to a particular situation.

The theory of soil mechanics is based on the assumption that the soil in question is homogeneous and isotropic throughout a given mass. Such idealism, however, is seldom realized in practice. The application of theory, therefore, must be tempered with judgement, and judgement can only come from experience. The application of theory, as presented in this book, incorporates the experience which the author has gained in the design of many buildings in a variety of soil situations.

Pictures, of course, are worth a thousand words. This book incorporates almost two hundred illustrations, each of which tells a story. Architects and engineers are visually oriented, and the story a picture tells becomes readily apparent to them. Because the text and the illustrations complement each other, it is recommended that both be studied in order to gain a clear understanding of the subject material.

It has been the author's goal to present the basic concepts and applications of soils and foundations in a clear, readable, and hopefully interesting way. The comments, suggestions and criticisms of readers will be appreciated.

A Brief Overview of Chapter Content

1. An introduction to the terminology, methodology and different standards used in the identification and classification of soils.
2. A discussion of the various physical properties of different kinds of soil and the test procedures used in their determination.
3. Information as to the different methods of field identification, sampling and engineering investigation of in-situ soils.
4. The different ways in which a footing may fail, and the theory by which the ultimate bearing capacity of a footing may be computed. Pressure bulbs and pressure distribution. The effect of ground water. The reasons for footing settlement, and methods by which settlement can be computed and evaluated.
5. A general consideration of spread footings, including excavation, formwork and reinforcing. A description of the different kinds of footings and the different situations in which each might be used. A discussion of the problems relating to the vertical and horizontal placement of a footing.
6. The theoretical and practical approach to the design and installation of piles, piers and caissons.
7. The theory behind lateral earth pressure. The way in which different soils exert different pressures. The equivilant liquid pressure theory. Practical methods by which lateral pressure can be analysed, both as to numerical value and effect.
8. Temporary retaining walls as required for major excavation. A general discussion of the architectural treatment of basement walls and retaining walls, including special strength considerations, such as prestressed rock and soil anchorages. Requirements relative to backfill and wall drainage systems.
9. The lateral pressure for which a wall must be designed. The different options of design. General reinforcing details. The different ways in which earth pressure can be transferred from the wall to the supporting element.

10. The need, theory and practical application of soil compaction, including a discussion of relative density, optimum moisture content and verification of in-place soil density. Included also are the compaction characteristics of the soils of the Unified Soil Classification System.
11. An in-depth discussion relative to the phenomenon of expansive clay. Recommendations regarding the construction of buildings and residences on sites consisting of expansive clay. Looking for evidence of damage.
12. The general classifications of rock, and the methods by which the load bearing characteristics of a rock mass can be determined.
    A. Transference of lateral load across a cold joint by the principles of shear-friction.
    B. Transference of lateral load across a cold joint by the use of shear keys. Recommendations regarding different shear key design loads.
    C. The ways in which the intensity of vertical pressure can be determined at any point in a soil mass due to the action of various types of loading.
    D. General recommendations relative to the empirical design and construction of slab on ground.
    E. Details relative to the transference of vertical load by the use of reinforcing dowels. Development lengths, as required for use in shear-friction analysis.
    F. The effect of ground water in terms of buoyancy on buildings.

Soils and Foundations
for
Architects and Engineers

1

Classification of Soils

1-1. GENERAL

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

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 of rock.

For a detailed discussion of rock the reader is referred to Chapter 12.

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, but may include grains of all four classifications in various proportions. Such soils are referred to as *mixed grained.*

Coarse Grained Soils

Coarse grained soils are defined as those soils whose individual grains are retained on a 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.

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.

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 which 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 a certain amount of 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 is 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, which are used to differentiate one clay type from another.

Colloids

Extremely small particles carry a surface charge of electricity. Such particles are called *colloids.* When a particle is in the colloidal state this surface charge exhibits a decisive influence on such properties as the plasticity and cohesion of the particles and the permeability of the soil mass. All clays are colloids.

For a detailed discussion of clays and colloids refer to Chapter 11.

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 upon 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.

Consistency

This is a term used to describe the degree to which a cohesive soil resists deformation. Consistency, therefore, relates to firmness. For a detailed discussion refer to Section 2-11.

Cohesion

Cohesion is a property by which particles of clay and plastic silt exhibit a measurable amount of stickiness, one to the other. It is this property by which these soils develop resistance to shear. For a detailed discussion of cohesion refer to Section 2-12.

Plasticity

Plasticity is the property of a soil which gives it the ability to be remolded and deformed without separating or breaking apart. All clay groups exhibit this ability, although not to equal extent. Silts also exhibit plasticity, but to a considerably less degree.

Atterberg Limits

Atterberg Limits define the four states of consistency for a cohesive soil in terms of water content. These four states are liquid, plastic, semi-solid, and solid. For a detailed discussion of the determination and use of these very important properties, refer to Section 11-6.

1-3. SOIL CLASSIFICATION

General

Various methods of soil classification have been devised as a means of attempting to standardize technical information from which the characteristics and load response of a given soil can be approximated. The general properties by which soils are classified are as follows:

1. Particle size
2. Particle distribution
3. Texture
4. The Atterberg Limits, of which the most important are the liquid limit and the plasticity index

Coarse grained soils are normally classified by particle size, distribution and texture. Fine grained soils are normally classified according to their properties as defined by the Atterberg Limits. It must be emphasized that all of these properties can only be determined by performing exacting tests in the controlled environment of a testing laboratory.

Classification Systems

There are four major systems of soil classification:

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

1-4. PARTICLE SIZE

General

The size and distribution of particles in a coarse grained soil are determined by performing a laboratory test called a *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 a 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 for Soils

This method provides for the quantitative determination of the size and distribution of particles within a soil mass. Soils retained on a No. 200 sieve are classified as coarse grained soils. Their properties are obtained by sieve analysis. Soils which pass a No. 200 sieve are classified as fine grained soils. Their properties are obtained by the sedimentation process.

Prior to 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 its separate grains with mortar and pestle.

Sieve Test

A standard sieve test, when performed on an oven dried coarse grained soil, 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 exclude 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. In order to perform the test, the sieves are vertically stacked in descending order of opening size. The prepared sample is placed on the upper sieve and the stack is attached to a mechanical shaker which imparts an upward and sidewise motion to the sieves. Each particle will fall through the stacked sieves until being retained on the sieve whose opening is smaller than the size of the particle. The

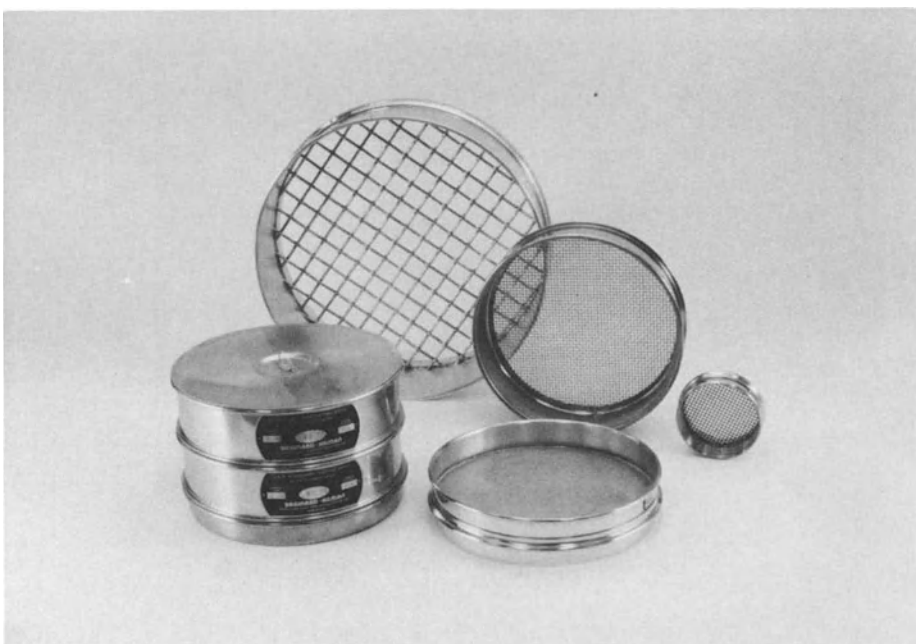


FIGURE 1-1. Standard testing sieves for particle size analysis. [Ref. 4]

larger particles are retained on the upper sieves, while the smaller particles are retained on sieves positioned lower in the stack.

All 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 number, which identifies the size opening within the mesh. The most frequently used sieves are the 3″, ¾″ and the Nos. 4, 10, 40, and 200. The clear width of the opening between the strands of the mesh is given in Table 1-1.

It must 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 upon the how the particle is aligned with respect to the sieve opening.

Classification by Particle Size

Particles are classified by name, according to the sieve upon which they are 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 the fine grain fraction of soils—silts and clays—do not depend solely on particle

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

Sieve Size	Millimeters	Inches[a]	Classification
			Cobbles
3″	75	3	
			Coarse gravel
¾″	19	¾	
			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		
			Silt or clay

[a] Sizes are approximate, and are for general interest only.

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. 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, and this weight is 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 1-2.

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 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 1-3.

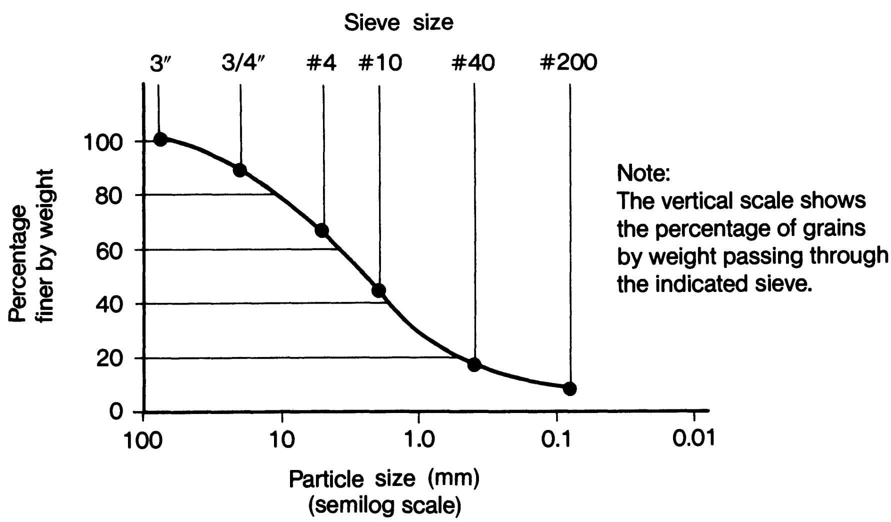


FIGURE 1-2. Typical particle distribution curve, as determined from sieve analysis.

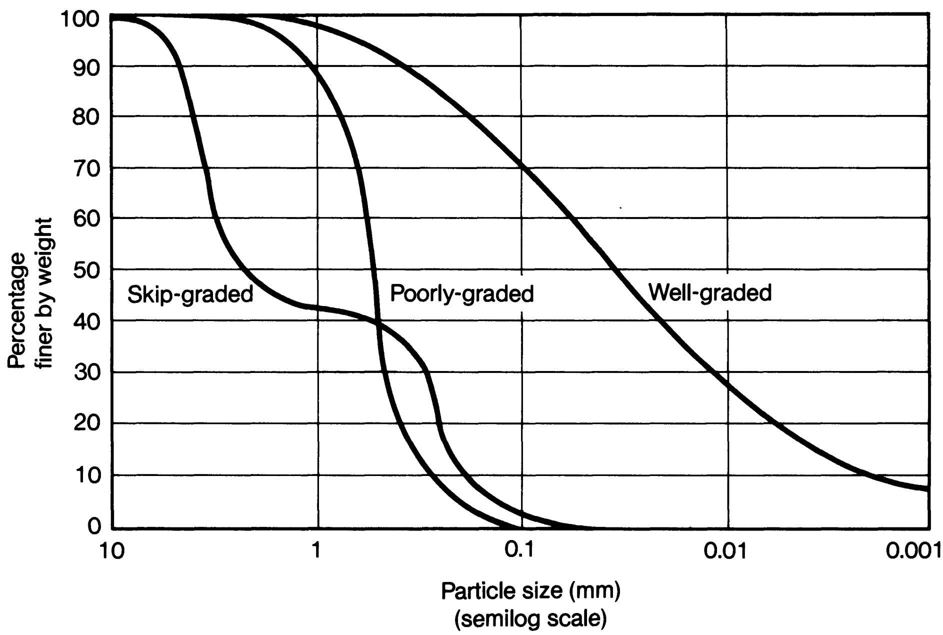


FIGURE 1-3. Particle distribution curves representative of skip-graded, poorly graded, and well-graded particles. [Ref. 10]

Coefficients of Uniformity and Curvature

When less than 5% of a soil identified as a sand or gravel passes a No. 200 sieve, it is evident 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 are determined as follows:

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

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

In order to determine the D values, it is first necessary to plot the particle distribution curve for the soil in question. This curve will be similar to the one illustrated in Figure 1-2. D values may then be taken from the curve. Each D value equals the grain diameter corresponding to the appropriate percent finer line on the curve.

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 conforming to the following limitations may be classified as well graded, those not conforming are classified as poorly graded:

$$C_u > 4 \quad \text{and} \quad 3 > C_c > 1 \quad \text{for gravel}$$

$$C_u > 6 \quad \text{and} \quad 3 > C_c > 1 \quad \text{for sand}$$

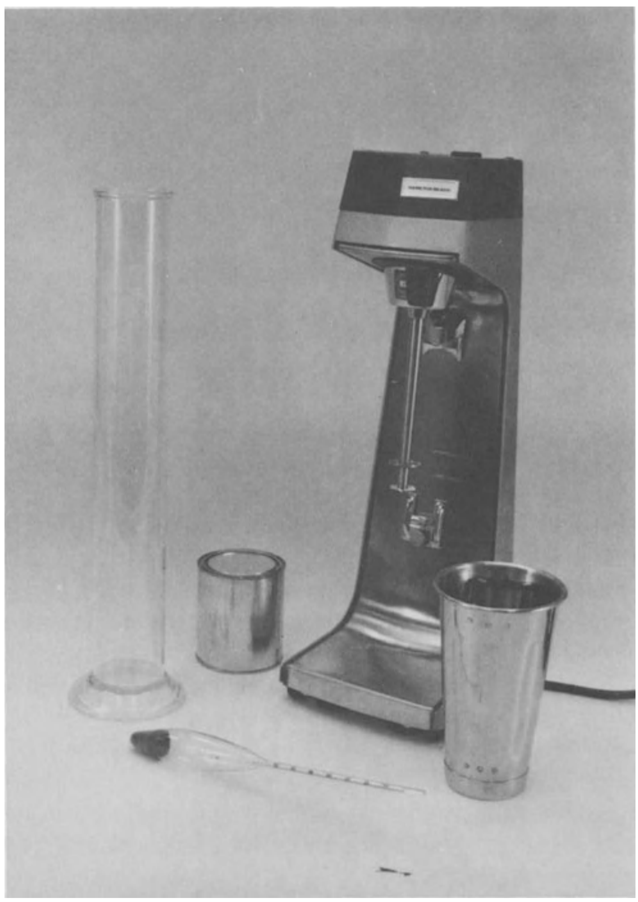


FIGURE 1-4. Hydrometer analysis set, as used to perform a sedimentation test on fine grained particles. [Ref. 4]

Sedimentation Test

Sedimentation tests, although to some degree approximate, are considered useful in determining the general size and distribution characteristics of particles within a fine grained soil. This test is performed by preparing a soil solution in distilled water, and allowing the soil grains to settle out without interruption. The equipment used in this test is the *hydrometer,* as illustrated in Figure 1-4.

The theory upon which this test is based is that the grains of soil will slowly settle out of the soil-water solution, with the heavier grains settling out first. The grains of soil weigh more than the water they displace. Therefore, when soil settles out of solution, the specific gravity of the remaining solution will slightly decrease. Time related readings which monitor the specific gravity of the solution at different times and at different depths provide an indication of the weight of soil remaining in the solution, and consequently the weight of soil which has settled out of solution. For detailed information relative to this rather complex test the reader is referred to ASTM Designation D-422.

1-6. SOIL CLASSIFICATION BY TEXTURE

The term *texture* refers to the classification of soil based solely on the distribution of the different particle sizes within the soil.

The textural classification system as illustrated in Figure 1-5 was developed by the United States Department of Agriculture. It is a very easy system to use and uses terms with which all architects and engineers are familiar. To use this system, the percentages by dry weight of sand, silt, and clay must be known. The intersection point of these three percentages is then found on the classification chart, 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 there is a substantial amount of gravel in the mix. 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 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 actuality.

One of the drawbacks to the correct use of this system is that the percentages of each constituent must be accurately known. In serious engineering, however, this is just one of the many things that must be done.

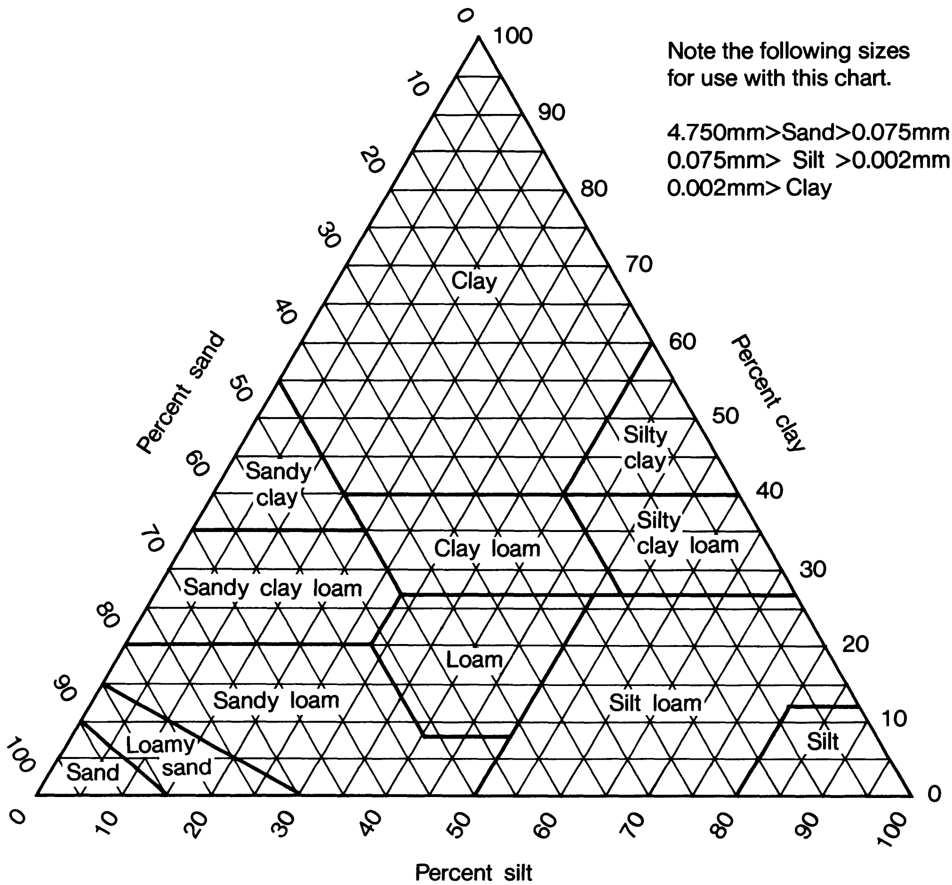


FIGURE 1-5. Textural classification of soils as developed by the United States Department of Agriculture. [Ref. 23]

1-7. AASHO CLASSIFICATION SYSTEM

AASHO Classification M-145, which originated with the Bureau of Public Roads, was ultimately adopted by the American Association of State Highway 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 though 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 as 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 as a standard reference in highway construction, this system of classification is not generally used by architects and engineers engaged in the design of buildings.

1-8. ASTM CLASSIFICATION SYSTEM

The American Society for Testing and Materials has developed the following system of soil classification based on the Unified Soil Classification System:

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

This system classifies soil into the fifteen basic groups which originated in the unified system. A somewhat different terminology is used, however, in the soil designation of each group. The system then subdivides these groups into a series of smaller groups according to the percentage of their constituents.

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 also on the behavior of the soil as characterized by plasticity.

This system, sometimes referred to as the USCS, has been adopted as standard by the Army Corps of Engineers and is the basis upon 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 two letters, one primary, the other secondary. 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 which are primarily coarse grained, and the characteristics of plasticity for those which are primarily fine grained. These letters are as described in Table 1-2.

The Fifteen Soil Classifications

A list of the fifteen divisions which are the result of this method of soil classification, including a general description of the soils and soil mixture in each division, is given in Table 1-3.

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
PI—Peat	
	For fine grained soils:
	L—Relatively low liquid limit
	H—Relatively high liquid limit

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 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.

Identification of Organic Soils

The first step in the process of soil identification is to determine whether the soil contains any organic material whatsoever. 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, backfill, etc. 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 to the soil a typically fibrous texture.
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 of an organic soil may be as high as several hundred percent, which far exceeds that found in most soils.
6. The specific gravity 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 test should be performed to determine 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 on the moist sample. When the same test is performed on an inorganic soil the plasticity index will vary only a few percentage points either up or down.

TABLE 1-3. USCS General Soil Groups. [Ref. 22]

Major Divisions			Symbol	General Description
Coarse Grained Soils, more than 50% of dry weight retained on No. 200 sieve	Gravels, greater % of coarse fraction retained on 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 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 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

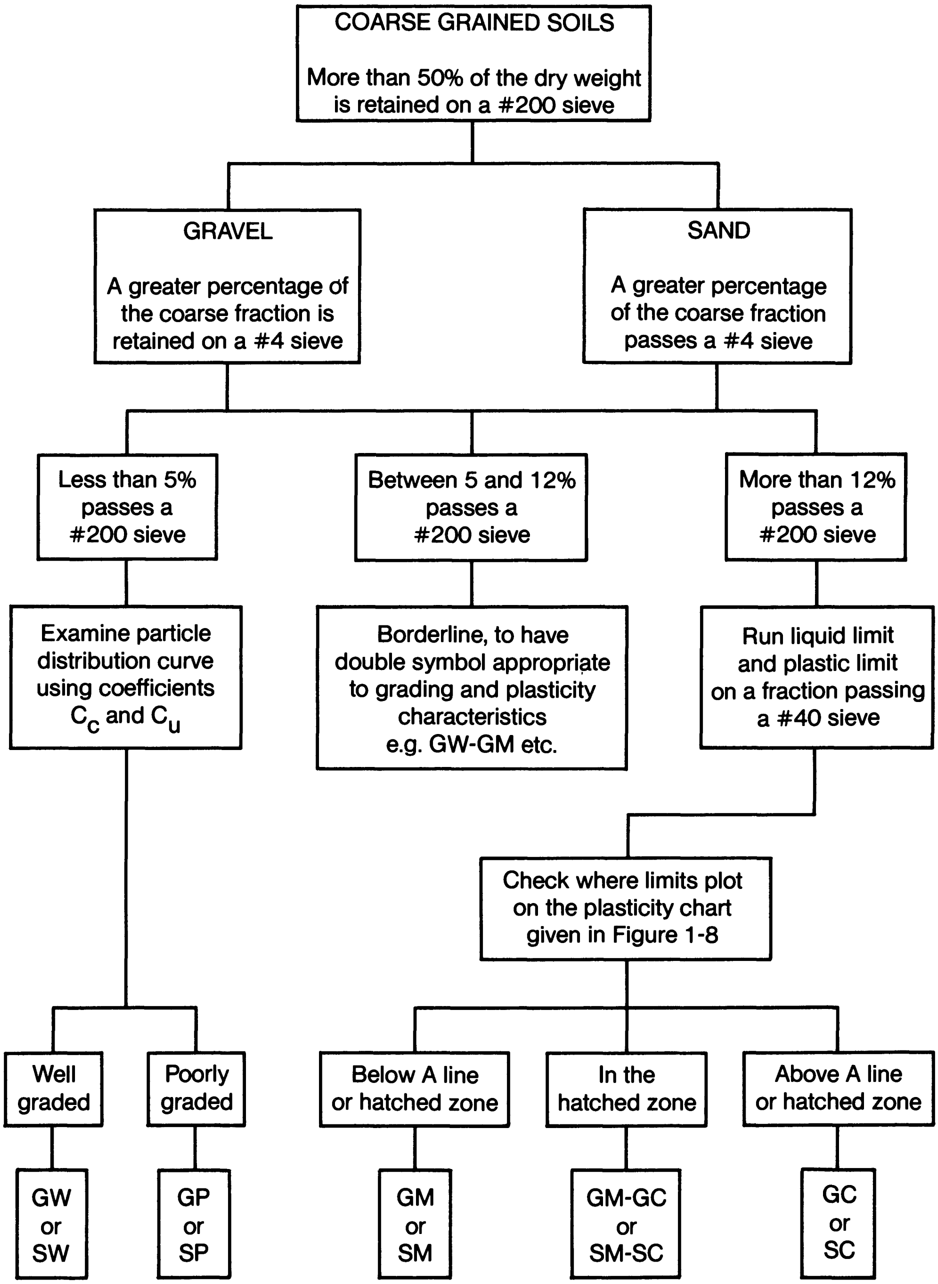


FIGURE 1-6. USCS Flow chart for identification of coarse grained soils. [Ref. 22]

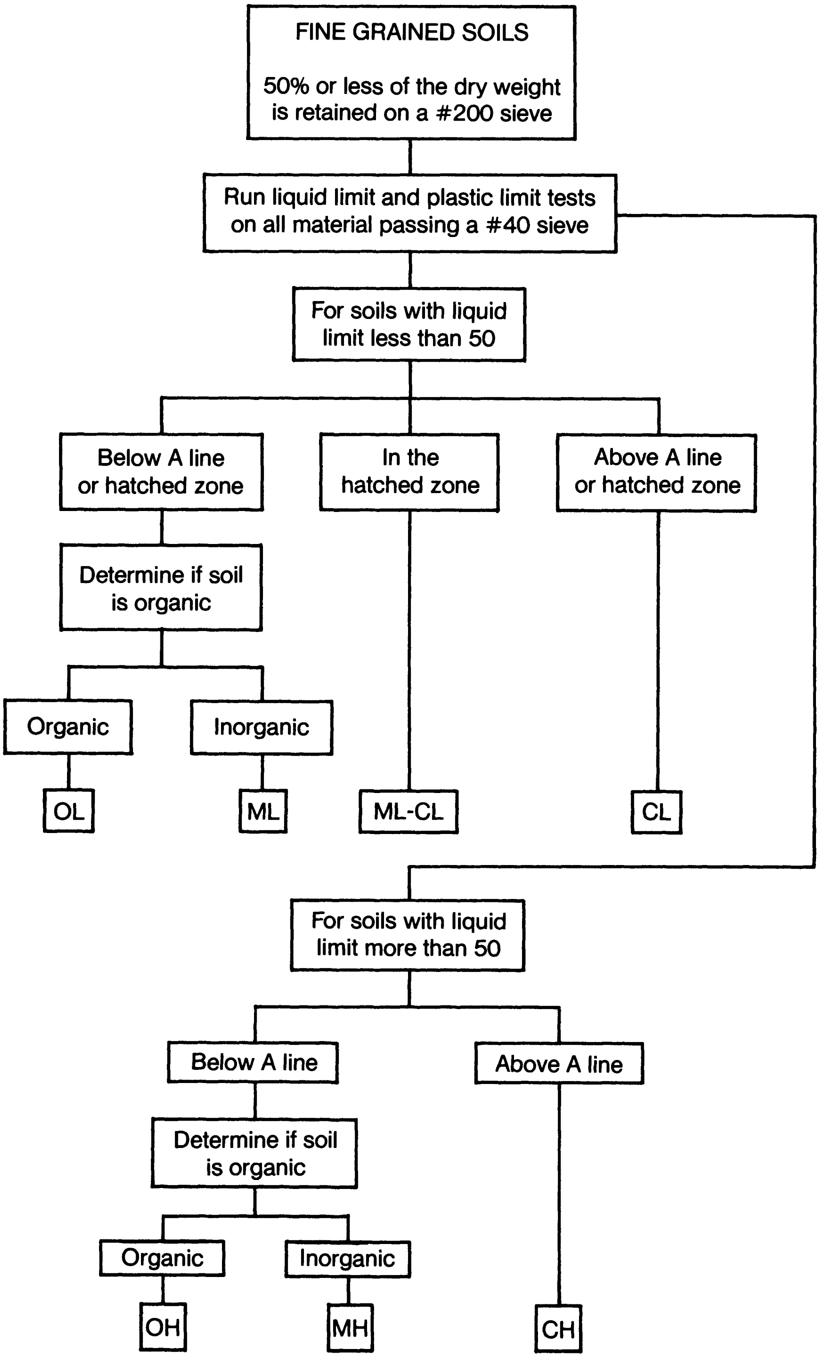


FIGURE 1-7. USCS Flow chart for identification of fine grained soils. [Ref. 22]

Identification of Inorganic Soils

Soils which do not contain organic material are classified as **inorganic** soils. Inorganic soils are grouped into two major classifications—coarse grained and fine grained. The determination into which of these groups a soil should be classified can usually be made by visual examination. In borderline cases, or in cases of uncertainty because of the presence of considerable mixed grained material, run a sieve analysis on a sample of soil, using a No. 200 sieve. A representative sample of the soil must be used, and must be oven dried prior to testing. Soils are classified as coarse grained when more than 50% of the dry weight of the sample is retained on the sieve. Soils are classified as fine grained when 50% or less of the dry weight of the sample is retained on the sieve. Coarse grained soils are further classified according to particle size and distribution. This procedure is shown in Figure 1-6. Fine grained soils are further classified according to plasticity, as shown in Figure 1-7.

Plasticity Chart

Prior to using the plasticity chart, as given in Figure 1-8, the liquid limit *LL* and the plastic limit *PL* of the soil in question must be determined by laboratory analysis. The plasticity index *PI* is then computed by the following formula:

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

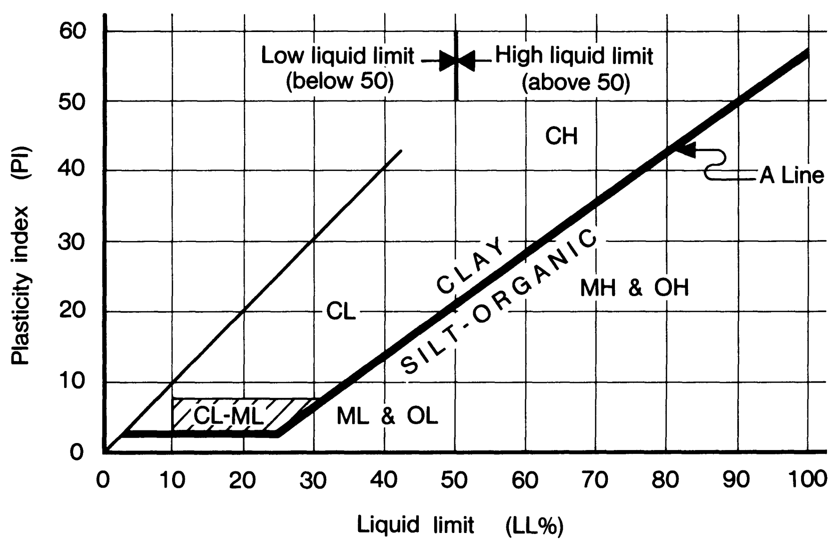


FIGURE 1-8. Plasticity chart for use in the identification of fine grained soils. [Ref. 22]

The chart is then entered with the known liquid limit and plasticity index. Identification is made depending upon where the soil 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, silts and organic soils plot below. The A line has been arbitrarily established according to the following formula:

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

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, soils below are ML. Those which fall within the hatched area are assigned the dual designation of CL-ML.

A secondary purpose in the use of this chart is to differentiate between soil groups GM and GC, and soil groups SM and SC. These are the groups which have more than a 12% fraction passing through a No. 200 sieve. Because of this relatively high amount 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 that fraction of the soil which passed 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 the proper soil identification can be made—those plotting below the A line 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. CLOSURE

It must be remembered that no system of classification is perfect, nor will any system answer all questions or solve all problems. It is the opinion of the author that soil classifications should be used only for the purpose of preliminary identification and comparison.

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, and the remaining 60% can be made up of silt and sand in any number of proportions. Clearly, this system cannot be used as the basis for serious foundation design.

The Unified Soil Classification System, 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 this is descriptive of: *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 judgement 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.

1-11. SAMPLE PROBLEMS

Example 1-1

Required: To identify a given soil according to the United Soil Classification System. Identification to be based on the sieve analysis of a sample of soil provided in Table 1-4.

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

(b) The coarse fraction (that which is larger than a No. 200 sieve) is computed in terms of a percentage of the total dry weight:

$$\frac{1800 - 72}{1800} \times 100\% = 96\%$$

In accordance with the flow chart on Figure 1-6, this soil is classified as coarse grained.

TABLE 1-4. Sieve Analysis.

Sieve No.	Weight Retained on Each Sieve	Percent of Total Dry Weight Passing
3″	0	100%
$\frac{3}{4}$″	324	82
No. 4	648	46
No. 10	252	32
No. 40	216	20
No. 200	288	4
Base	72	0
	1800 gm	

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

$$\frac{324 + 648}{1800 \times 0.96} \times 100\% = 56\%$$

Because this percentage is greater than 50%, this soil is 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 1-9.

(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:

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

These values are then compared to the limits set forth for a well graded gravel:

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

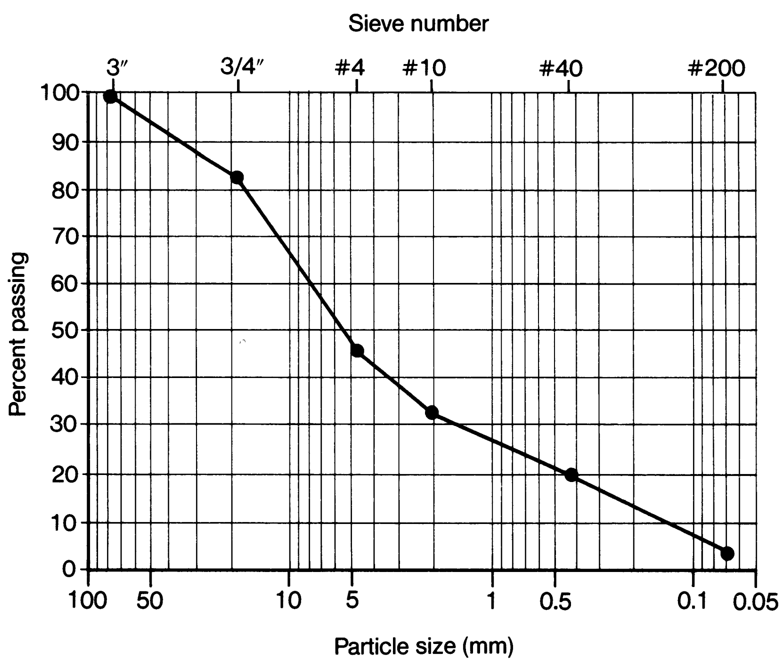


FIGURE 1-9.

Since these limits are satisfied, the soil should be classified as follows: *GW Well-graded gravels, gravel-sand mixtures, little or no fines.*

Example 1-2

Required: To identify a given soil according to the United Soil Classification System. Identification to be based on the sieve analysis of a sample of soil provided in Table 1-5.

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

(b) The coarse fraction (that which is larger than a No. 200 sieve) is computed as a percentage of the total dry weight:

$$\frac{1800 - 54}{1800} \times 100\% = 97\%$$

In accordance with the flow chart on Figure 1-6, this soil is classified as coarse grained.

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

$$\frac{144 + 540}{1800 \times 0.97} \times 100\% = 39\%$$

Because this percentage is less than 50%, the soil is 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 1-10.

(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:

TABLE 1-5. Sieve Analysis.

Sieve No.	Weight Retained on Each Sieve	Percent of Total Dry Weight Passing
3″	0	100%
$\frac{3}{4}$″	144	92
No. 4	540	62
No. 10	594	29
No. 40	126	22
No. 200	342	3
Base	54	0
	1800 gm	

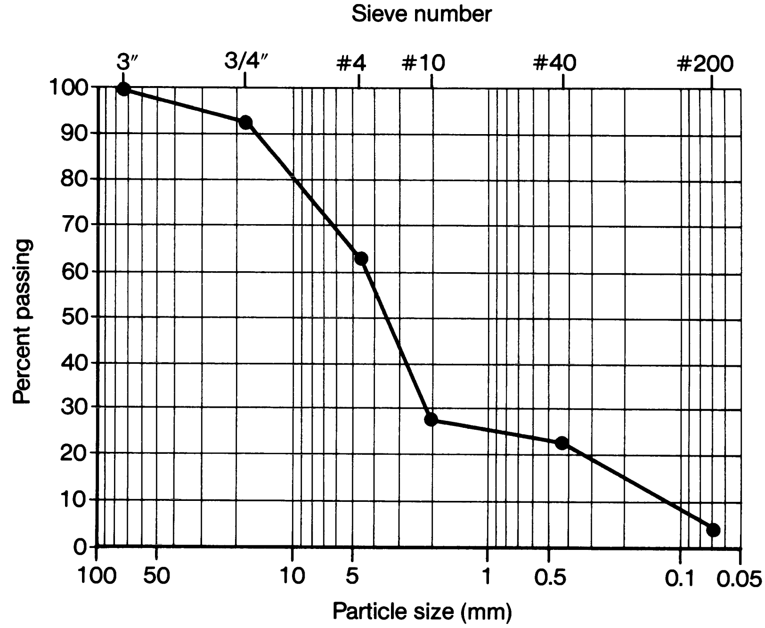


FIGURE 1-10.

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

These values are then compared to the limits set forth for a well graded sand:

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

Because these limits are not satisfied, the soil should be classified as follows: *SP Poorly graded sands or gravelly sands, little or no fines.*

Example 1-3

Required: To identify a given soil according to the United Soil Classification System. Identification to be based on the sieve analysis of a sample of soil provided in Table 1-6.

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

(b) The coarse fraction (that which is larger than a No. 200 sieve) is computed as a percentage of the total dry weight:

$$\frac{1800 - 272}{1800} \times 100\% = 86\%$$

TABLE 1-6. Sieve Analysis.

Sieve No.	Weight Retained on Each Sieve	Percent of Total Dry Weight Passing
3″	0	100%
$\frac{3}{4}$″	198	89
No. 4	450	64
No. 10	468	38
No. 40	162	29
No. 200	270	14
Base	252	0
	1800 gm	

In accordance with the flow chart on Figure 1-6, this soil is classified as coarse grained.

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

$$\frac{198 + 450}{1800 \times 0.86} \times 100\% = 42\%$$

Because this percentage is less than 50%, this soil is 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:

$$\begin{aligned} \text{Liquid limit } LL &= 42\% \\ \text{Plastic limit } PL &= \underline{15\%} \\ & \ 27\% = \text{Plasticity index } PI \end{aligned}$$

(f) The numerical values of liquid limit and plasticity index are plotted on the plasticity chart of Figure 1-8. Because the intersection point of these values falls above the A line, the soil should be classified, according to Figure 1-6, as follows: *SC Clayey sands, sand-clay mixtures.*

Example 1-4

Required: To identify a given soil according to the United Soil Classification System. Identification to be based on the sieve analysis of a sample of soil provided in Table 1-7.

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

TABLE 1-7. Sieve Analysis.

Sieve No.	Weight Retained on Each Sieve	Percent of Total Dry Weight Passing
3″	0	100%
$\frac{3}{4}$″	0	100
No. 4	72	96
No. 10	126	89
No. 40	306	72
No. 200	144	64
Base	1152	0
	1800 gm	

(b) The coarse fraction (that which is larger than a No. 200 sieve) is computed as a percentage of the total dry weight:

$$\frac{1800 - 1152}{1800} \times 100\% = 36\%$$

In accordance with the flow chart on Figure 1-7, this soil is classified as fine grained. A liquid limit and plastic limit evaluation must now be made on the soil fraction passing a No. 40 sieve.

(c) A laboratory analysis provides the following information:

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

(d) The numerical values of liquid limit and plasticity index are plotted on the plasticity chart of Figure 1-8. Because the intersection point of these values falls below the A line, the soil should be classified as follows: *ML Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.*

Note: A plasticity index evaluation in accordance with Table 11-2 indicates that this particular soil has a medium shrink-swell potential.

Example 1-5

Required: To reevaluate the soil of Example 1-4, given a different laboratory analysis:

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

(d) The numerical values of liquid limit and plasticity index are plotted on the plasticity chart of Figure 1-8. Because the intersection point of these values falls above the *A* line, the soil should be classified as follows: *CH Inorganic clays of high plasticity, fat clays.*

Note: A plasticity index evaluation in accordance with Table 11-2 indicates that this particular soil has a relatively high shrink-swell potential.

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. The laboratory analysis should be performed in accordance with the following ASTM Standard:

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 can be illustrated visually as shown in Figure 2-1, where V represents volume, W represents weight, and the subscripts a, w, and s represent air, water, and solids.

The general procedure by which the weight and volume are determined is itemized below:

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

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.

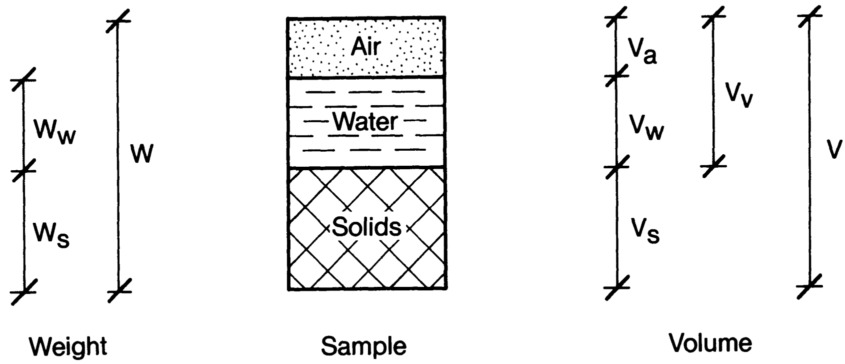


FIGURE 2-1. Weight-volume relationship of air, water and the solid constituents of a typical soil sample.

Step 3. The weight of the solid constituents W_s must now be determined. The sample is first oven dried at a constant temperature of 105 to 115 degrees Centigrade. This will drive off all of the free water from the sample. If there are any clay particles in the sample the drying process will also remove the absorbed water molecularly bonded to those particles. For a discussion of absorbed water refer to Article 11-2. The sample which remains after the drying process consists solely of solid constituents, whose weight can now be determined.

Step 4. The weight of water W_w originally contained in the sample can be determined by subtracting the weight of the solids from the initial weight of the sample:

$$W_w = W - W_s$$

Step 5. The volume of water V_w, corresponding to the weight of water found in Step 3, may now be computed. Remember that density is the ratio of weight to volume, and that the density of water is 62.4 pcf; therefore:

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

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.

The weights and volumes obtained by these measurements can be used to determine important physical properties of the in-situ soil from which the undisturbed sample was obtained.

2-2. UNIT WEIGHT, DENSITY

Unit weight and *density* are terms which are synonymous, and may be used interchangeably. These terms, symbolized by γ, are used to express the ratio of total weight to total volume, and are computed numerically in pounds per cubic foot.

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

All soils contain air, water, and solids. The unit weight of any soil, therefore, can be indicated thus:

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

where V_v is the volume of voids.

The unit weight of any volume of soil can be dramatically altered by varying the amount of water contained within it. This is because any volume of water will weigh less than an equal volume of the solid constituents of the soil. Unit weights corresponding to three different water contents are of importance to the soils engineer. These three conditions are illustrated in Figure 2-2. Note that although the weight of water varies the weight of solids remains the same.

Sample 1. This sample is representative of the in-place soil, which normally consists of air, water and solids. This unit weight is symbolized by γ.

Sample 2. 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 weight is called the *saturated weight,* and is symbolized by γ_{sat}. When the saturated weight of the in-place soil must be determined, the required procedure can be performed on the original

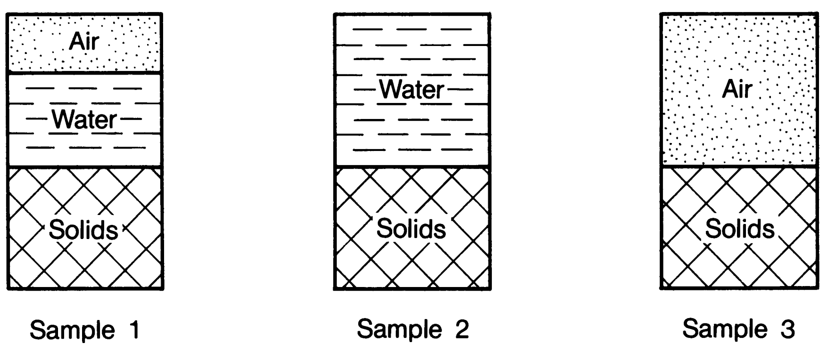


FIGURE 2-2. The effect of variation in water content on any given sample of soil.

sample after step 2 and before step 3, or it can be performed on a different sample. In either case water must be added to the sample until all of the voids are completely filled. The sample is then weighed. This weight divided by the initial volume will give the saturated unit weight.

Sample 3. This sample is representative of the condition which occurs when all of the void normally filled with water has been replaced with air. The weight of this sample, therefore, is its least possible weight. This weight is called the *dry weight,* and is symbolized by γ_{dry}.

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

2-3. POROSITY

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

$$n = \text{porosity} = \frac{V_v}{V} \qquad (2\text{-}2)$$

Porosity is a property which depends on the physical characteristics of the soil particles, including their size, shape and uniformity, and their 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 in that it is an indication of the permeability of the soil.

2-4. VOID RATIO

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

$$e = \text{void ratio} = \frac{V_v}{V_s} \qquad (2\text{-}3)$$

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:

$$e = \text{void ratio} = \frac{n}{1 - n} \qquad (2\text{-}4)$$

By substituting $V - V_s$ for V_v, Formula (2-3) can be rearranged to equate V_s, V, and e in another useful way:

$$V_s = \text{volume of solids} = \frac{V}{e+1} \tag{2-5}$$

When the void ratio is known this formula can be used to determine the volume of solids without performing the test specified in Step 6.

2-5. WATER CONTENT

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

$$w\% = \text{water content} = \frac{W_w}{W_s} \times 100\% \tag{2-6}$$

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,* which expresses the volume of water to the volume of voids:

$$S\% = \text{degree of saturation} = \frac{V_w}{V_v} \times 100\% \tag{2-7}$$

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:

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

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

$$G_s = \frac{W_s}{62.4V_s} \tag{2-8}$$

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:

G_s for sands and gravels: 2.65 to 2.68

G_s for silts and clays: 2.58 to 2.75

Specific gravity can be used to develop relationships between weights, water content and degree of saturation, as follows:

$$e = \text{void ratio} = \frac{V_v}{V_s} \times \frac{V_w}{V_w} = \frac{V_w}{V_s S} = \frac{W_w/62.4}{(W_s/62.4G_s)S}$$

Therefore:

$$e = \frac{W_w G_s}{W_s S} \tag{2-9}$$

and

$$e = \frac{wG_s}{S} \tag{2-10}$$

When using Formula (2-10) it is noted that the water content w must be expressed as a decimal fraction.

2-7. REPRESENTATIVE VALUES OF PHYSICAL PROPERTIES

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

TABLE 2-1. Physical Properties of Typical Soils in Natural State. [Ref. 20]

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

2-8. RELATIVE DENSITY OF COARSE GRAINED SOILS

A coarse grained soil, it will be recalled, is one which consists primarily of sand and gravel, either without fines entirely or with 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.

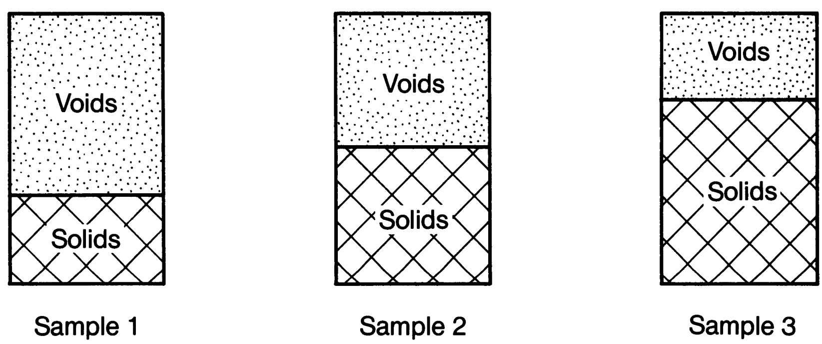


FIGURE 2-3. The effect of variation in void ratio on any given sample of coarse grained soil.

Sample 1. This sample is representative of a very loosely packed soil, having maximum voids. The void ratio is e_{max}.
Sample 2. This sample is representative of the in-place 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}.

A soil in its loosest possible state will have minimum density and maximum void ratio. A soil in its densest possible state will have maximum density and minimum void ratio. *Relative density* is a term used to numerically compare 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. Relative density can be expressed in terms of void ratio, as indicated:

$$\text{Relative density} = D_r = \frac{e_{max} - e_{nat}}{e_{max} - e_{min}} \times 100\% \qquad (2\text{-}11)$$

Where:

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

By examining the upper and lower limits of Formula (2-11) 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 (2-11) it is necessary to determine the three void ratios illustrated in Figure 2-3. 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 Section 2-1. Void ratios may then be computed in accordance with Section 2-4. The sample representing the loosest possible state is prepared by allowing the soil to gently free fall into a container of known volume. The sample representing the densest possible state is prepared by packing the soil into the container until the container will accept no more soil. The void ratio of the soil in its natural state must be determined on an undisturbed sample.

The numerical accuracy of the three required void ratios depends upon 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 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.

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

Description	D_r, %	Density, pcf
Loose	<35	<90
Medium dense	35 to 65	90 to 110
Dense	65 to 85	110 to 130
Very dense	>85	>130

Relative density is of primary importance in problems involving compaction of coarse grained soils. For information on this subject, and for a different method by which relative density can be determined, refer to Section 10-6.

2-9. 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 4, and to lateral earth pressure, as developed in Chapter 7.

This angle is a measurement of the ability of a particular soil to resist shear through intergranular friction. The magnitude of this angle depends on several factors, including the kind of soil, the size, shape, and distribution of the grains, 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 possess a measurable angle of internal friction, the magnitude of which decreases as the amount of fines becomes more dominant. The angle of sandy silt, for example, usually varies between 35 to 20 degrees, whereas the angle of silty clay or sandy clay usually varies between 20 to 0 degrees. A soil consisting almost solely of clay will have no angle of internal friction.

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 moisture content and degree of compaction that will exist during service, otherwise such tests will be valueless. In the absence of a laboratory analysis the angle of internal friction can be approximated from the general information given in Table 2-3.

The angle of internal friction can also be approximated from the test borings because there is a correlation between it and the blow count N, as recorded during the standard penetration test. This test is described in Section 3-5, and the correlation between angle and blow count is given in Figure 2-4.

TABLE 2-3. 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°

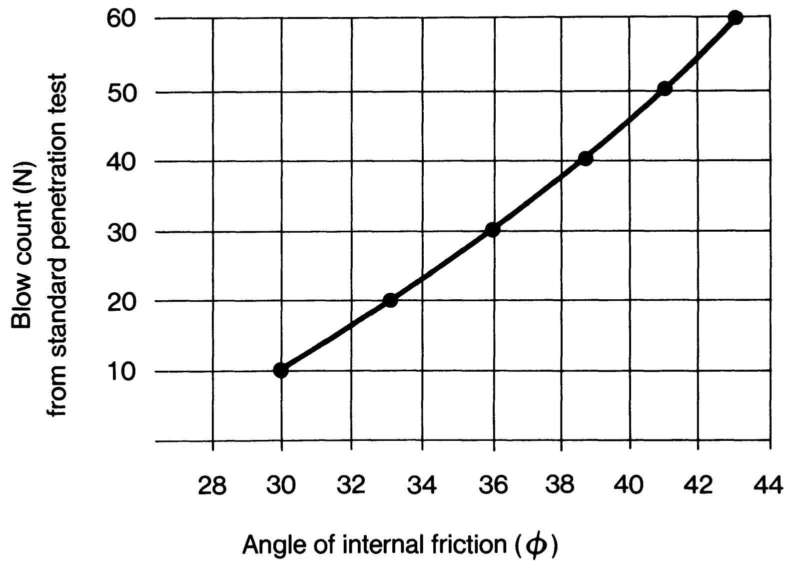


FIGURE 2-4. Approximate correlation between the angle of internal friction and the blow count N, for coarse grained soils. [Ref. 16]

The degree of compaction of a coarse grained soil is sometimes referred to in terms of *blow count.* Soils with blow counts between 0 to 4 are referred to as very loose, those between 4 to 10 are loose, between 10 to 30 are medium, 30 to 50 are dense and greater than 50 are very dense.

2-10. 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 one and one-half to two times its diameter. The sample is subjected to a compressive force of increasing intensity until failure occurs. This procedure must be performed quickly, otherwise moisture may permeate out of the sample, in which case the test is invalidated. The unconfined compression strength is found by dividing the load at which the sample breaks by its cross sectional area.

All procedures in this analysis must be performed in accordance with the following ASTM Standard:

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

Typical apparatus used in this test procedure is shown in Figure 2-5.

The sample used for the unconfined compression strength test can be used again in the test for sensitivity, as described in Section 2-13. The only proviso is that the moisture content of the sample must remain unchanged throughout both testing procedures.

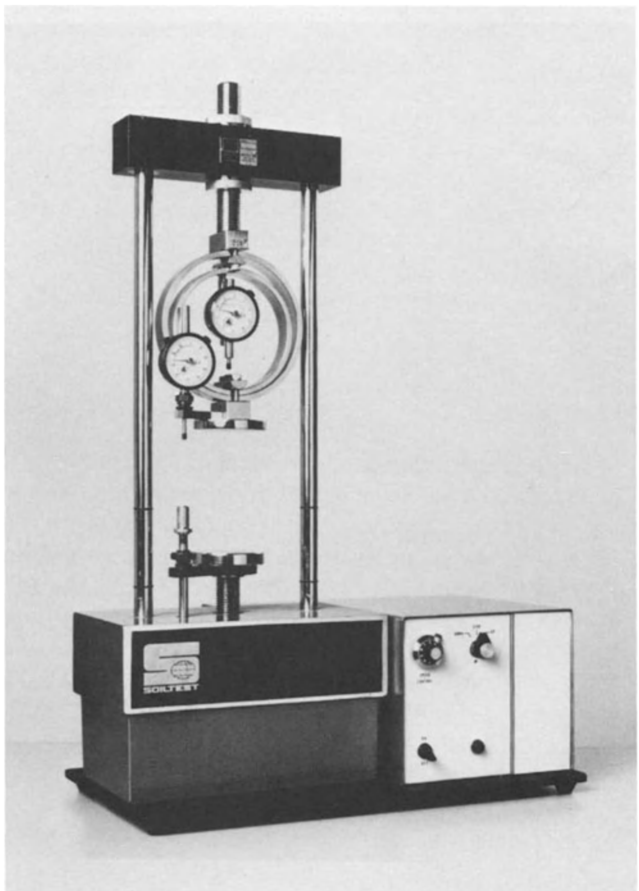


FIGURE 2-5. Unconfined compression apparatus, as used to determine the unconfined compression strength of a cohesive soil.

2-11. 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 reasonable correlation between consistency, unconfined compression strength, and the blow count N, as described in Section 3-5. This correlation is shown in Table 2-4.

TABLE 2-4. Correlation of Physical Properties for Cohesive Soils. [Ref. 16]

Consistency	Field Examination	N	q_u, tsf
Very soft	Easily penetrated several inches by fist	Under 2	Under 0.25
Soft	Easily penetrated several inches by thumb	2 to 4	0.25 to 0.50
Medium	Can be penetrated several inches by thumb with moderate effort	4 to 8	0.50 to 1.00
Stiff	Readily indented by thumb but penetrated only with great effort	8 to 15	1.00 to 2.00
Very stiff	Readily indented by thumbnail	15 to 30	2.00 to 4.00
Hard	Indented with difficulty by thumbnail	over 30	over 4.00

2-12. COHESION

Cohesion is a measurement of shear and is a property inherent to clay. It may also be exhibited to a somewhat lesser degree by mixed-grain soils consisting predominantly of clay. Cohesion is the property whereby a soil fraction has the ability to resist shear independently of any normal pressure which may exist on the plane of rupture. Cohesive soils, therefore, develop shear resistance in a completely different way than 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 much like a glue which produces resistance to shear by bonding the surfaces together. The numerical value of cohesion is usually taken as being equal to one-half of the unconfined compression strength q_u of the soil.

2-13. SENSITIVITY

Sensitivity 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 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 which 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:

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

Commonly used descriptions relating to sensitivity are given in Table 2-5.

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 sample. A sensitivity of 16, on the other hand, indicates that the remolded sample has experienced a considerable loss in strength.

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, as illustrated in Figure 8-1. 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

TABLE 2-5. Sensitivity of Clays. [Ref. 13]

S_t	Description
>16	quick
8 to 16	extra sensitive
4 to 8	sensitive
2 to 4	normal

that its volume is very large compared to that of the solid particles. 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 also 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 instances, 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 installation of timber or steel piles by driving

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.

2-14. SAMPLE PROBLEMS

Example 2-1

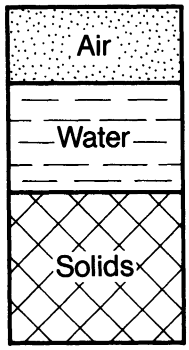
Required: To determine the physical properties of a particular sample of a coarse grained soil. This is an undisturbed sample, and is representative of an in-place soil deposit. Refer to Figure 2-1 and perform the indicated measurements and calculations:

Step 1. Volume V of container is 1 CF.
Step 2. Weight W of soil sample is 117.6 lb.
Step 3. Weight W_s of solids of the oven dry sample is 105.0 lb.
Step 4. Weight W_w of water is $117.6 - 105.0 = 12.6$ lb.
Step 5. 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:

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

$$\text{Volume } V_a \text{ of air in sample} = 1.000 - 0.202 - 0.630 = 0.168 \text{ CF}$$

These results are illustrated in Figure 2-6.



Volume (cf)	Weight (#)
0.168	0.0
0.202	12.6
0.630	105.0
1.000	117.6

FIGURE 2-6.

The physical properties of this soil are now computed, as indicated:

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

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

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

$$w = \frac{12.6}{105.0} \times 100\% = 12.0\% \tag{2-6}$$

$$S = \frac{0.202}{0.370} \times 100\% = 54.6\% \tag{2-7}$$

$$G_s = \frac{105.0}{62.4 \times 0.630} = 2.67 \tag{2-8}$$

Example 2-2

Required: To determine the saturated and dry unit weights of the soil previously analysed in Example 2-1. Refer to Figure 2-2 and make the indicated measurements and calculations:

Sample 1. The unit weight of the in-place soil is 117.6 pcf.

Sample 2. 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 saturated weight of the sample, is, therefore:

$$105.0 + 0.370 \times 62.4 = 128.1 \text{ pcf}$$

Sample 3. In Step 3 all of the water was removed from the sample, leaving only the solids, whose weight is 105.0 lb. The dry unit weight, therefore, is 105.0 pcf.

Example 2-3

Required: To determine the relative density D_r of the soil previously analysed 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 2-1 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	oven dried sample
Step 6.	V_v	0.474 CF	0.320 CF	
	V_s	0.526 CF	0.680 CF	
	e	0.901 max	0.471 min	from Formula (2-3)

The relative density of the in-situ soil of Example 2-1 can now be computed using Formula (2-11):

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

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

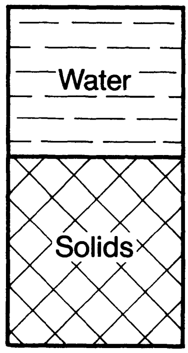
Relative density can also be computed by using unit weights instead of void ratios. This alternate procedure is described in Section 10-6 of Chapter 10.

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-place soil deposit. Refer to Figure 2-1 and perform the indicated steps.

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

Step 1. Volume V of container is 1 CF.
Step 2. Weight W of soil sample is 113.4 lb.



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

FIGURE 2-7.

Step 3. Weight W_s of solids of the oven dry sample is 82.7 lb.
Step 4. Weight W_w of water is $113.4 - 82.7 = 30.7$ lb.
Step 5. Volume V_w of water is $30.7/62.4 = 0.492$ CF.
Step 6. Water measurement is not required because soil is saturated:

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

$$\text{Volume } V_a \text{ of air in sample} = 0.0$$

These results are illustrated visually in Figure 2-7.
The physical properties of this soil are now computed, as indicated:

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

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

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

$$w = \frac{30.7}{82.7} \times 100\% = 37.1\% \quad (2\text{-}6)$$

$$S = 100\% \quad \text{(saturated soil)} \quad (2\text{-}7)$$

$$G_s = \frac{82.7}{62.4 \times 0.508} = 2.61 \quad (2\text{-}8)$$

Example 2-5

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

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

From Formula (2-8):

$$V_s = \frac{106.4}{62.4 \times 2.68} = 0.636 \text{ CF}$$

from which

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

From Formula (2-6):

$$W_w = 0.201 \times 106.4 = 21.4 \text{ lb}$$

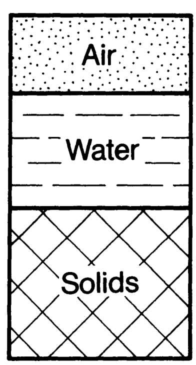
from which

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

and

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

These results are illustrated visually in Figure 2-8.



Volume (cf)	Weight (#)
0.021	0.0
0.343	21.4
0.636	106.4
1.000	127.8

FIGURE 2-8.

The density of the in-place soil therefore, is 127.8 pcf.
From Formula (2-3):

$$e = \frac{0.364}{0.636} = 0.572$$

From Formula (2-7):

$$S = \frac{0.343}{0.364} \times 100\% = 94.2\%$$

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 his engineer, should make a preliminary investigation of the site. The purpose of this investigation is to acquaint both the architect and the 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 then tests must be made 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 then 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. Such a study 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.

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, or 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 ultimate 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 been already been made, then, of course, the architect 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. There are several lightweight, portable drilling tools which are available to him for that purpose. These tools are easily operated by one man and can be conveniently transported and stored in the trunk of a car.

The *Iwan type auger,* sometimes referred to as a *post hole digger,* is a hand operated drilling tool used to recover samples of earth in soils that are sufficiently cohesive so that the side walls of the drill hole will not collapse into the hole. This auger is usually equipped with a cutting blade having a three inch diameter. With favorable soil conditions, samples can be recovered from depths 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* is a drilling tool operated much like the Iwan type auger. It differs from that auger, however, in construction. The drilling end of the ship auger is an open spiral screw having a usual diameter of two inches. As the auger is lifted out of the ground, the material through which it was drilled is caught within the spiral and brought to the surface. Although this auger can be drilled through granular material it can only recover samples of cohesive soil. The operation of a ship auger is illustrated in Figure 3-1.

FIGURE 3-1. Ship auger, used in procuring samples of soil by hand. [Ref. 1]

FIGURE 3-2. A backhoe in the process of cutting a trench for visual examination of a soil profile. [Ref. 7]

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 3-2.

The advantage of using a back-hoe rather than digging a test pit by hand or by clam shovel is that this kind of 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. After making sure that the side walls will not collapse, the architect may elect to go down into the trench in order to 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. No open holes or softly filled holes should ever be permitted to remain overnight or even unattended for any time whatsoever.

3-2. PRELIMINARY IDENTIFICATION OF SOILS

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 of each of these soils 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 which are pressed between the thumb and forefinger. Fragments that are predominately 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.

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/hrs
Sandy clay	low/high	slow/none	medium	sec/hrs
Silty clay	med/high	slow/none	medium	min/hrs
Clay	high	none	tough	hrs/days
Organic silt[a]	low/med	slow	weak	min/hrs
Organic clay[a]	med/high	none	tough	hrs/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.

Thread Test

This 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 the 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.

3-3. FIELD SURVEY

If the owner of the property has recently purchased it, a field survey will have been made immediately prior to the time of legal transfer. Such a survey would have been required by 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 *metes and bounds*
   b. A layout of all existing barriers and obstructions, including those that are of natural origin and those that are man made
   c. A layout of all known underground or overhead utilities

3-4. ENGINEERING INVESTIGATION

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 and 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 in order to monitor the depth to ground water for an extended period of time
3. Certain field tests, such as one of several different available tests to determine the relative density of coarse grained soils, and the shear strength of fine grained soils
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 investigation company under contract with the owner. This company must work closely with the project architect and engineer to insure 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.

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 determine the following characteristics of the underlying soils or rock formations:

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 q_u
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 q_u
25. RDQ (rock quality designation)

3-5. TEST BORINGS

General

Test borings are holes drilled into the ground for the purpose of exploring the soils and rock formations which underlie the site of a proposed building. Test borings should normally be placed in a square grid measuring approximately fifty feet on a side, and the grid should extend approximately twenty-five feet beyond the extremities of the building. Borings should extend into the earth approximately twenty 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 and depend on the kind of information required and whether the drilling is done in soil or in rock. All drilling procedures require heavy equipment, an example of which is illustrated in Figure 3-3.

Earth Borings

The purpose of an earth boring is to make a small vertical hole in the ground to obtain a sample of earth from the soil directly below the bottom of the hole. There are two standard procedures by which this hole can be made.

In the first method, a section of hollow pipe called a *casing* is driven into the ground by a falling weight. By adding sections and repeating the process, the boring can be extended into the ground to whatever depth is desired. In the second method, a continuous flight hollow stem auger is advanced into the ground by rotary drilling. This is the same auger that would be used to obtain samples of rock. The soil within the hole is periodically removed and brought to the surface by using a machine-driven auger or by washing it out with pressurized water. Material brought to the surface by auger can be visually examined and will yield some information about its characteristics. Material surfaced by washing is very disturbed and is usually valueless for examination, except to distinguish between soils rich in sands and soils rich in clay.

Samples of soil existing beneath the bottom of the hole are obtained by driving a sampling device into the ground. After the sampling device has been driven into the ground far enough to be filled with earth, it is then brought to the surface and the sample recovered. There are two kinds of samples—representative and undisturbed.

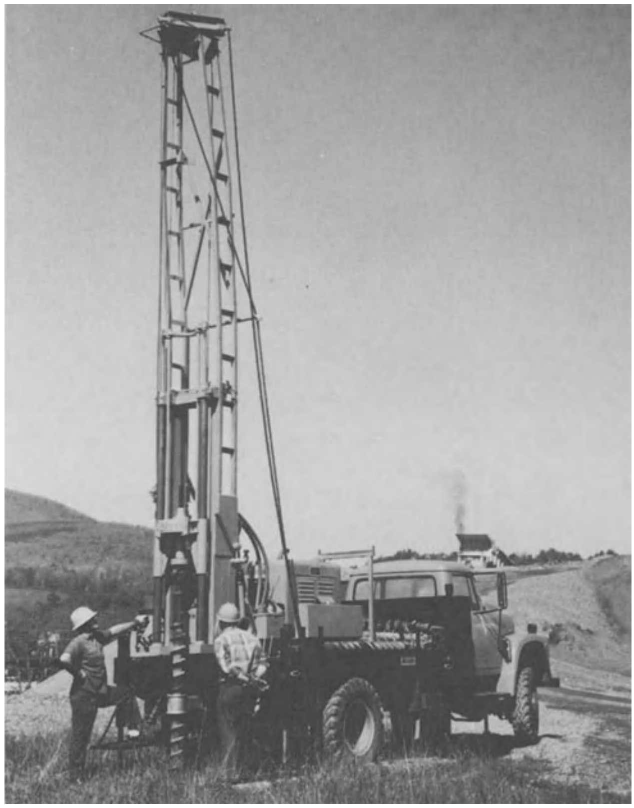


FIGURE 3-3. A truck transported drilling rig in the process of advancing a continuous flight auger to refusal. [Ref. 1]

Representative 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 3-4.

The split tube sampler is designed to open into two halves, thereby exposing the entire sample. Samples obtained by the split tube method are not undisturbed samples. They are representative of the material found at the bottom of the hole, but they do not represent the soil in-situ. These samples are therefore of limited value for cohesive soils and are used primarily for the general identification of the soil. Their real value is with granular soils, where laboratory tests do not require undisturbed samples.

Split tube samplers are available in several different sizes. The one most frequently used has a 2″ outside diameter, a 1⅜″ inside diameter, and a clear length within the barrel of 18″. In all probability, the main reason for the popularity of this particular sampler is that it is the one specified to be used in the Standard Penetration Test, as described in a later paragraph.

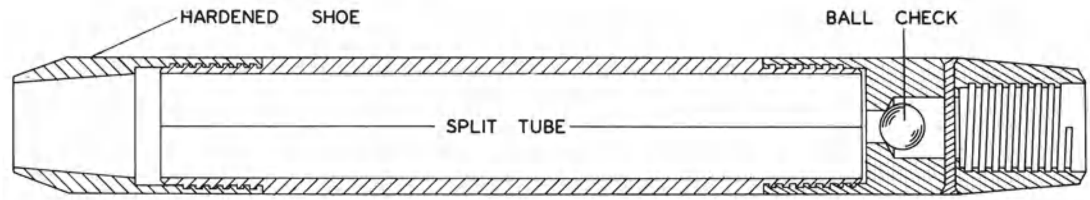


FIGURE 3-4. A split tube sampler, used for obtaining disturbed samples of granular and cohesive soils. [Ref. 1]

Undisturbed samples are required for the laboratory analysis of cohesive soils. 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 3-5.

Thin wall tube samplers have steadily gained in popularity since their development, and are now considered to be the industry standard for obtaining undisturbed samples. Different sizes of thin wall tube samplers are available. The tube most frequently used has a 3″ outside diameter, a 2⅞″ inside diameter and a clear length within the barrel of 18″. These tubes are usually made of 11 gage seamless steel tubing.

With each kind of sampler it is important to protect the undisturbed sample from loss of water during transport. 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. The sample is then transported in its original liner to the testing laboratory for whatever analysis is required. The sample can also be transported in a

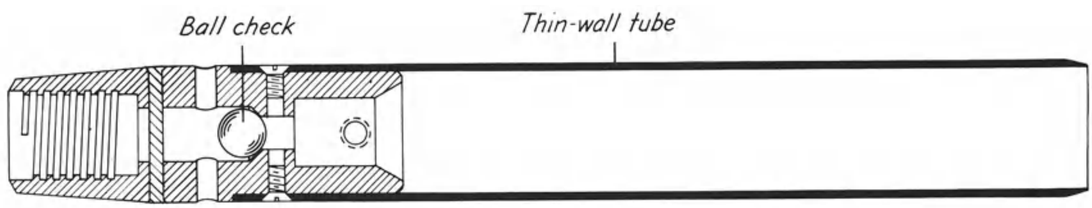


FIGURE 3-5. A thin wall tube sampler, used for obtaining undisturbed samples of cohesive soils. This sampler is frequently called a Shelby Tube. [Ref. 1]

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.

dry-ice refrigeration box which 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. In this test, a two inch diameter split tube sampler is driven into the ground just below the bottom of the bore hole with a one hundred and forty pound hammer having a free fall of thirty inches. In driving, a record is kept of the number *N* of blows required to advance the sampler a measured distance of twelve inches. Table 3-2 gives the terminology usually associated with the penetration resistance as determined by the test.

Depth to Bedrock, Refusal

In certain situations a decision is made to carry the foundations down to bedrock, as described in Section 12-10. In this situation the primary purpose of the test boring program is to determine the depth and contour of the surface of the bedrock. This can readily be determined during the standard penetration test by extending the casing or the continuous flight auger until refusal.

Refusal may be defined as the point at which continued driving of the casing or auger does not result in any further measurable penetration.

When founding on bedrock the characteristics of the rock must be determined. Refer to Section 3-7 and to Chapter 12 regarding this determination.

Water Table, Perforated Pipe

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

1. Sands and clays exhibit different characteristics depending upon whether they are above or below the water table.
2. All occupied parts of the building below the water table must be waterproofed.
3. High water table or deep excavation could cause serious problems with buoyancy, even to necessitating a change in the architectural design of the building. Refer to Appendix F for information regarding the phenomenon of buoyancy.

In order to monitor any variation in the depth of the water table over an extended period of time, 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 of 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.

3-6. TYPICAL TEST BORING LOG

The log of a typical test boring is shown in Table 3-3. The information given therein is indicative of the type of information found in most test boring logs. Note the following:

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

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

3-7. CORE BORINGS

The foundations for a building must sometimes bear on 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.

TABLE 3-3. Boring Log of Hole No. 8A2S-2958, Fort Polk. [Ref. 22]

Samples	Depth	Description		*N*
	0			
A		GRAVEL	fill, loose, moist	
	2.5			19
B		CLAY	low plasticity, soft, moist, sandy throughout, reddish brown to rust	
	5.0			9
C				
	7.5			10
D		SAND	fine to coarse grained, loose to medium dense, moist, clayey with soft reddish brown seams	
	10.0			15
E	12.5			12
	15.0	Water ▼		11
F	17.5			10
	20.0			9
G	22.5			2
	25.0	SAND	fine to coarse grained, loose to medium dense to dense, wet, clayey throughout, with soft light gray to yellowish brown seams noted throughout, tan to gray to rust with reddish brown zones	3
H	27.5			6
	30.0			4
I	32.5			4
	35.0			4
J	37.5			5
K	40.0	CLAY	low to medium plasticity, soft to stiff, wet, gray	22
L	42.5			24
	45.0	SAND	fine to coarse grained, loose to medium dense, wet, clayey, tan to light gray to rust in zones	10
M				
	47.5			35
N		CLAY	low plasticity, soft, wet	
	50.0			

2. The weight of the building is so massive that adequate resistance in bearing can not be provided except by the 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 at the point of bearing, and for some distance beneath. Core borings, which are a procedure by which samples of the

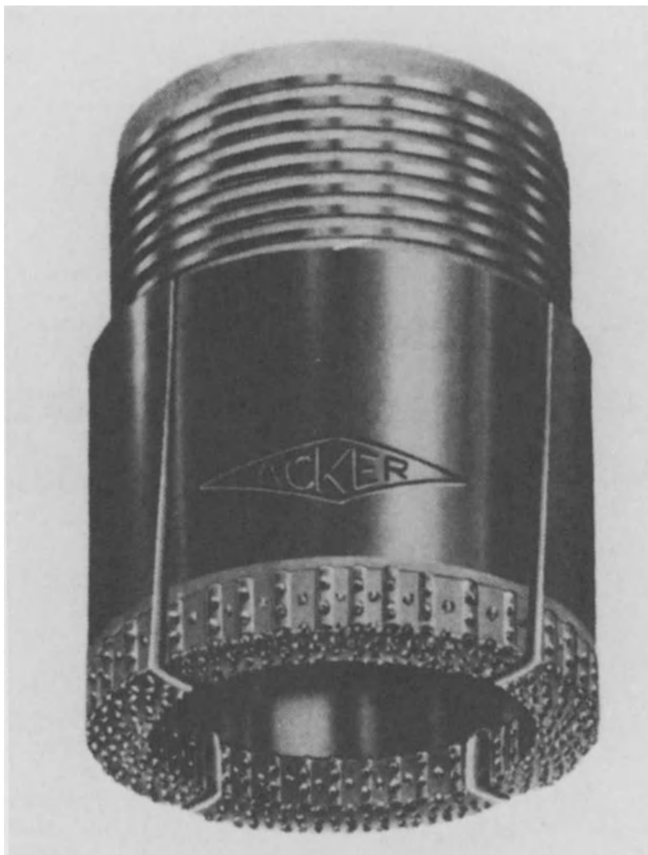


FIGURE 3-6. A standard diamond core bit, as used for obtaining samples of hard rock. [Ref. 1]

TABLE 3-4. Standard Core Diameters.[a].

Outside Diameter of Core Barrel	Diameter of Core Recovered
$1\frac{1}{2}$	$\frac{7}{8}$
$1\frac{7}{8}$	$1\frac{1}{8}$
$2\frac{3}{8}$	$1\frac{5}{8}$
$2\frac{15}{16}$	$2\frac{1}{8}$

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

rock are recovered and brought to the surface for evaluation, are used for this purpose.

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 bedrock. The drill rod, which has been fitted with a non-core 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 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.

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 pictured in Figure 3-6.

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 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, an example of which is shown in Figure 3-7.

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.

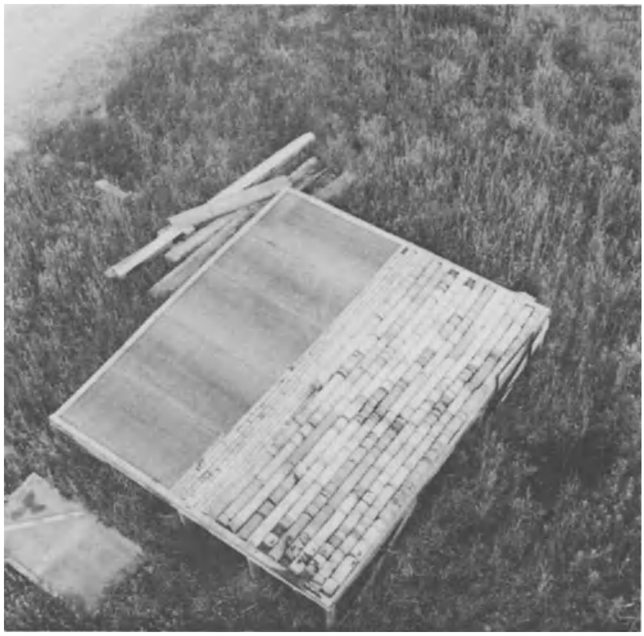


FIGURE 3-7. A typical core box, used to transport rock cores. [Ref. 1]

Core samples are ultimately delivered to the testing laboratory where the strength and quality of the rock is evaluated, and bearing pressures are established. For an in-depth discussion of the testing and evaluation of rock refer to Chapter 12.

3-8. GEOLOGIC DESCRIPTION OF SITE

Test borings show only those subsurface features which 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 3-8. 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.

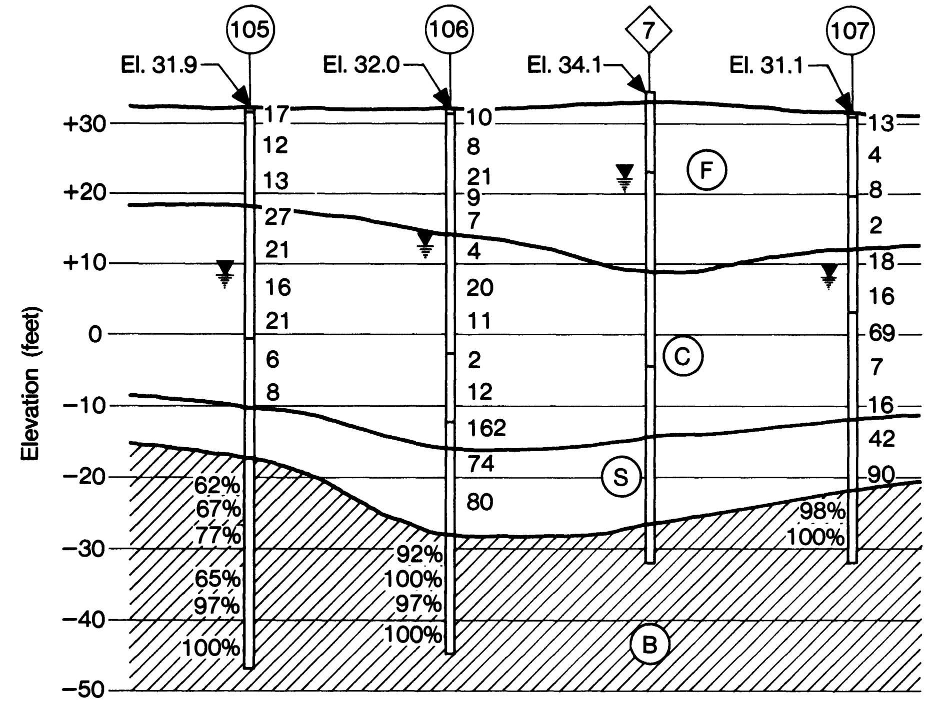


FIGURE 3-8. A geologic section taken through the site of one of the author's projects.

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.

3-9. SHEAR TESTS

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 important, therefore, to accurately determine the shear strength of soils situated in close proximity to the proposed construction. The procedures used for this determination are:

1. Field Test. This is called a *vane shear test,* and can be performed only on soils which exhibit a measurable cohesion.
2. Laboratory Tests. The *direct shear test* and the *triaxial compression test* are tests performed in the laboratory on an undisturbed sample of soil. These tests are very versatile because they can be used to test not only cohesive soils but also soils having little or no cohesion.

Vane Shear Field Test

Vane shear testing 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 3-9.

The vane shear test is performed on cohesive soils in the field in accordance with the following ASTM Standard:

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

This test must be carried out in cased test boring holes. To perform this test the vane is pushed into the soil below the casing, being 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 having different diameters and lengths.

Because of its inherent accuracy and simplicity of operation vane shear testing

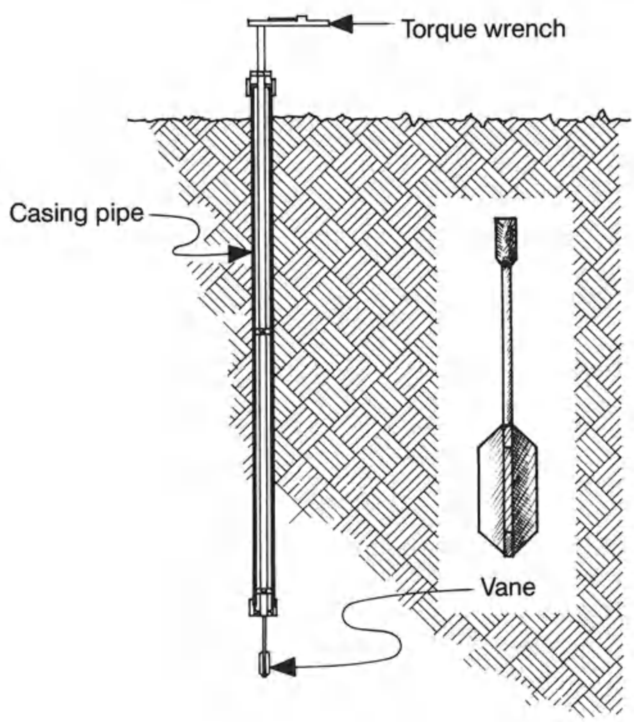


FIGURE 3-9. Vane shear apparatus, as used to determine the shearing strength of an in-situ cohesive soil. [Ref. 1]

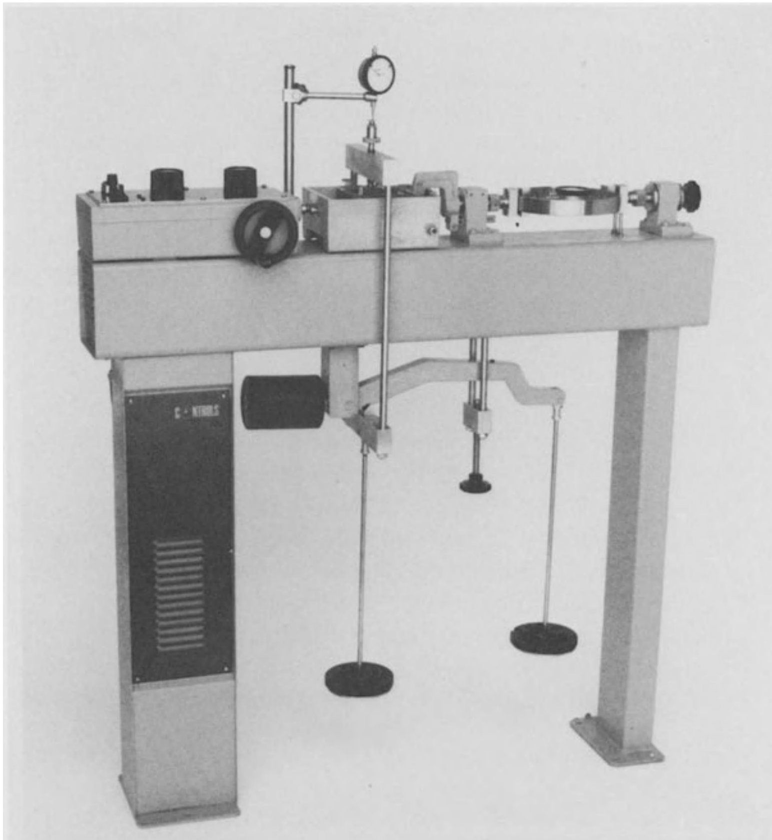


FIGURE 3-10. Direct shear apparatus, as used to determine the shearing resistance of soils in the laboratory. [Ref. 4]

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.

Direct Shear Laboratory Test

The direct shear test is performed in the laboratory on undisturbed samples of soil. The soil can be coarse, fine or mixed grain. All work must be performed in accordance with the following ASTM Standard:

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

The type of apparatus used for this test is shown in Figure 3-10.

In this test, the soil sample 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 shearing 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 shearing 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 shearing stress s are computed. This test should be repeated in its entirety at least two more times. The results of these tests can then be plotted, using the normal stress on the abscissa and the shearing stress on the ordinate. The resulting graph can be used to determine the unit cohesion and the angle of internal friction of the soil. For a graph representative of several different soils refer to Figure 7-3.

Triaxial Compression Test

For this test the soil sample is 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. Because this test can be performed on non-cohesive soils as well as on cohesive soils, the sample must be encased in a flexible membrane before it is installed into the cylinder. The type of apparatus in which this test is performed is illustrated in Figure 3-11.

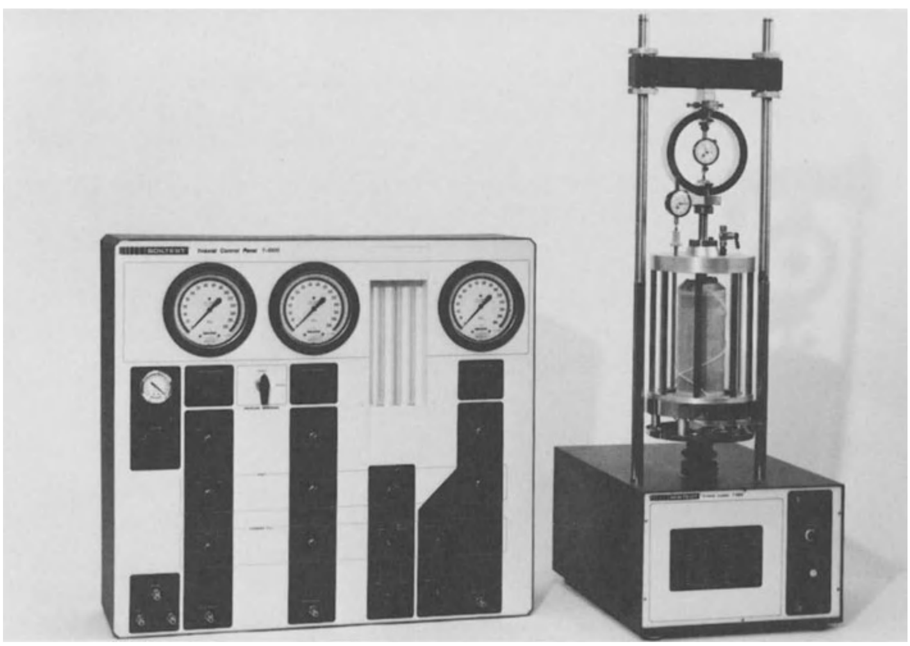


FIGURE 3-11. A triaxial test set, which may be used as a direct shear test or as an unconfined compression test. [Ref. 17]

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

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

The test is usually conducted by holding the confining pressure constant while steadily increasing the vertical pressure through the action of an axial load. The pressures occurring at the time of failure are used to calculate, through the use of Mohr's Circle, the corresponding normal and shearing stresses. This test should be repeated on several different samples, after which a graph of the interdependency of these two stresses can be plotted.

The end results of the triaxial test are essentially the same as those of the direct shear test. One reason, however, why the triaxial test may be preferred over the direct shear test is that ultimate failure of the sample need not occur at a predesignated plane, but will occur at any plane of inherent weakness in the sample.

It may be of interest to note that when a triaxial compression test is run on a sample while the confining pressure is omitted, the test becomes nothing more than that of an unconfined compression test. Such a test, however, can only be run on a soil which exhibits a measurable cohesion.

3-10. CLOSURE

A state of the art, cost effective structural design meeting and enhancing the many architectural requirements, is the proper goal of every professional engineer on every project. This goal cannot be achieved without secure knowledge of the soil upon which the building will bear. The architect bears the responsibility of overseeing a preliminary investigation of the site coupled with a preliminary identification of the soils within the zone of interest. This work is frequently done by the architect long before an engineer is brought into the project. The engineer's task, then, is to determine what further investigations, as described in Section 3-4, are necessary to insure the achievement of the overall goal.

4

Allowable Soil Bearing Pressure

4-1. GENERAL DESIGN CONSIDERATIONS

It is very difficult to accurately predict 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 cost effectively designed unless the engineer can combine the proper mixture of theory, experience, and intuition into all of 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 supports it. The determination of this allowable bearing pressure requires the in-depth consideration of two completely different situations:

1. The literal failure of the soil to support the imposed load, the result of which is that the footing breaks into the ground
2. Settlement which when excessive will cause severe damage to the structure itself and to any elements attached to it

These design considerations will be discussed in detail in the paragraphs which follow.

4-2. 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 4-1, 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, as illustrated in Figure 4-1. Note the falling in of the earth on one side of the footing and the pushing up of the earth on the other side.

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 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

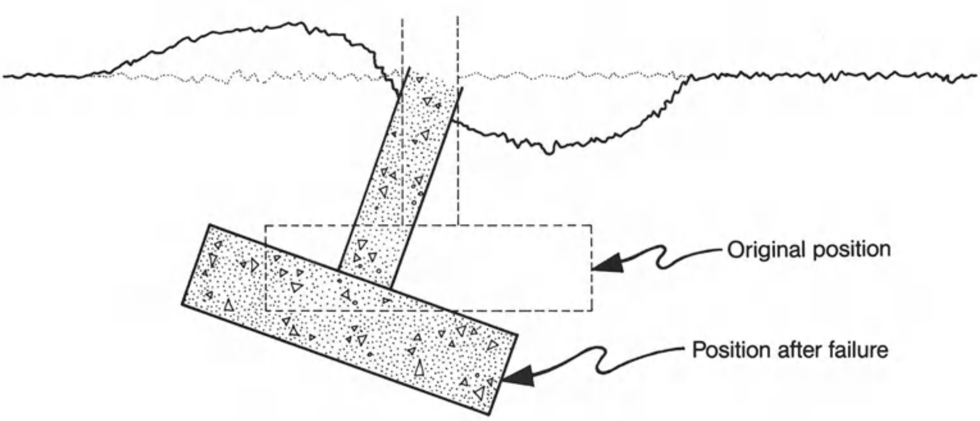


FIGURE 4-1. Footing failure due to breaking into the soil.

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.

4-3. 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 which 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 to the design of the foundations of a building continuously pays off in sound, cost effective, and prudent engineering.

The variables upon which the ultimate bearing capacity is dependent are cohesion, surcharge, soil density and the shape of the footing. These variables are mathematically expressed as follows:

q_d is the ultimate bearing capacity of the soil at the base of the footing, psf

c is the cohesion of the soil, psf

B is the width of a continuous footing, the minimum width of a rectangular footing, or the side of a square footing, feet

R is the radius of a round footing, feet

a_1 and a_2 are shape factors for use with rectangular footings, whose numerical values are given in Table 4-1

γ is the density of the in-situ soil, pcf, obtained by direct measurement, guidelines of which are given in Table 2-1

D_f is the depth of the footing below the lowest adjacent ground surface, feet

N_c, N_q, and N_γ are 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 Figure 4-2.

$$N_c = (N_q - 1) \cot \phi \qquad \text{[Ref. 8]}$$

TABLE 4-1. 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

(The term containing N_c considers the effect of cohesion.)

$$N_q = e^{\pi \tan\phi}\left[\tan^2(45° + \phi/2)\right] \qquad \text{[Ref. 8]}$$

(The term containing N_q considers the influence of surcharge.)

$$N_\gamma = (N_q - 1)\tan(1.4\phi) \qquad \text{[Ref. 8]}$$

(The term containing N_γ considers the density of the soil and the width of footing.)

Due to the many intangibles involved, the author suggests the use of the follow-

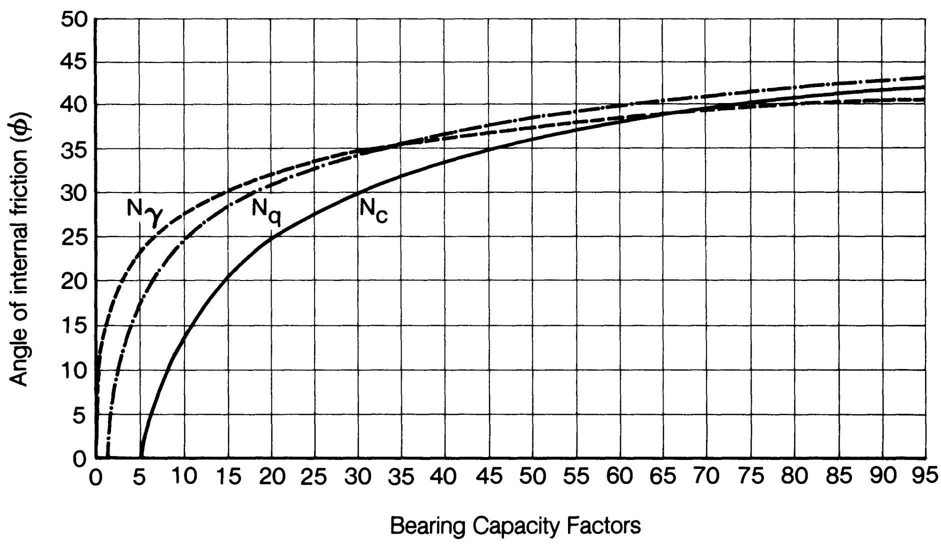


FIGURE 4-2. Bearing capacity factors for shallow footings as a function of the angle of internal friction. [Ref. 8]

ing equation for N_γ, as a somewhat conservative approximation of the referenced equation:

$$N_\gamma = 1.5(N_q - 1) \tan \phi$$

In the above formulas ϕ is the angle of internal friction, as discussed in Sections 2-9 of Chapter 2 and 7-2 of Chapter 7.

The angle of internal friction of a mixed grained soil can only be accurately determined by a laboratory analysis. The accuracy of the bearing capacity factors, therefore, is likewise dependent on 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 predominantly 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 2-4 of Chapter 2. For a purely granular soil the angle of internal friction can be approximated from Table 2-3 of the same chapter.

The numerical value of cohesion can be determined by laboratory analysis when highly accurate values are required. Under normal circumstances, however, cohesion is assumed equal to one-half of the unconfined compression strength q_u, whose value is determined in the laboratory or which can be approximated by correlation to blow count, as given in Table 2-4.

The development of the ultimate bearing capacity of soil beneath a continuous footing is quite different from that of soil beneath a square or rectangular footing. The former is more a case of two dimensional development, whereas the latter is more a case involving all three dimensions. The desirability of using one general equation, even though in modified form, has lead to the use of empirical shape factors which compensate for the difference in the ultimate bearing capacity of footings having different shapes.

Footings on Mixed Grained Soil

The four general equations for the ultimate bearing capacity of a footing bearing on a mixed grained soil are as follows:

Continuous Footing:

$$q_d = 1.0\ cN_c + \gamma D_f N_q + 0.5B\gamma N_\gamma \tag{4-1}$$

Square Footing:

$$q_d = 1.2\ cN_c + \gamma D_f N_q + 0.4B\gamma N_\gamma \tag{4-2}$$

Round Footing:

$$q_d = 1.2\ cN_c + \gamma D_f N_q + 0.6R\gamma N_\gamma \tag{4-3}$$

Rectangular Footing:

$$q_d = a_1 c N_c + \gamma D_f N_q + a_2 B \gamma N_\gamma \tag{4-4}$$

It should be noted that the bearing capacity formulas applicable to both continuous and square footings can be derived from Table 4-1 by using the formula for a rectangular footing with the proper a_1 and a_2 values.

Footings on Granular Soil (Cohesion c = 0)

Pure sand and pure gravel are granular materials which have no cohesion. The resistance to penetration of the footing depends, therefore, solely on the interlocking and actual physical contact between the particles. The numerical value of this resistance is a function of the angle of internal friction of the soil beneath the footing. When computing the ultimate bearing capacity of a footing bearing on a purely granular material, the first term in the general equation, that which enumerates the effect of cohesion, should be deleted. The second and third terms should remain.

Footings on Cohesive Soil (Bearing factor $N_\gamma = 0$)

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 purely granular material, the third term in the general equation, that which enumerates the effect of the density of the soil, should be deleted. The first and second terms should remain, although it must be noted that the effect of the second term will be very small, and could be omitted without inducing significant error.

4-4. ALLOWABLE SOIL BEARING PRESSURE

The four preceding equations are used to compute, in psf, the ultimate bearing capacity of the soil at some predetermined depth below the surface of the earth. The allowable bearing pressure for which the footing may be designed is a function of this ultimate bearing capacity, the depth of the footing below grade and the safety factor, all as indicated in the following formula:

$$q_a = \frac{q_d - \gamma D_f}{SF} \quad \text{in psf} \tag{4-5}$$

Where:

q_a is the net allowable soil bearing pressure for which the footing may be designed, based on the area of contact between the soil and the footing

q_d is the ultimate bearing capacity, as determined from Equations (4-1) through (4-4)
γD_f is the weight of the soil above the base of the footing
SF is the safety factor

Note that the ultimate bearing capacity is reduced by the weight of the 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 as a reasonable minimum. Some codes permit less, but remember—the architects and engineers are responsible for their work, not the writer of the code.

4-5. PRESSURE DISTRIBUTION—PRESSURE BULBS

All footings impart pressure to the soil beneath it. As an introduction to the subject of pressure distribution it must be understood that 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

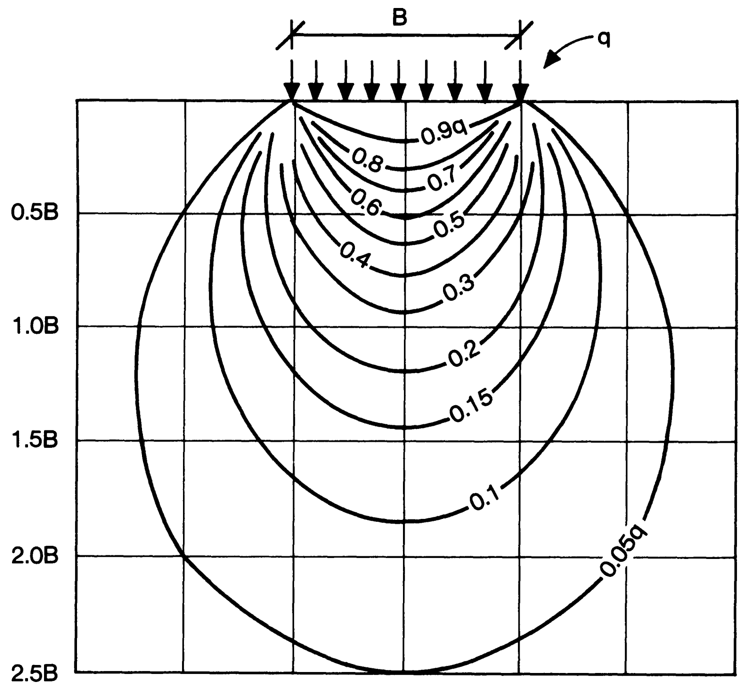
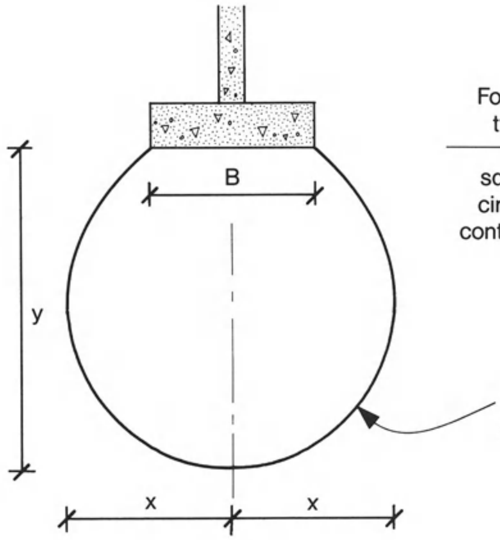


FIGURE 4-3. A typical pressure bulb, as produced by a circular footing. [Ref. 19]



Footing type	Assumed depth	Assumed 1/2 width
square	y = 2 B	x = 1 B
circular	y = 2 B	x = 1 B
continuous	y = 6 B	x = 2 B

FIGURE 4-4. 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.

diminishing intensities. This effect is called the *pressure bulb effect,* and is illustrated in Figure 4-3.

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, 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 balloon. The bulb of a circular footing has been illustrated in Figure 4-3, and that 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 and settlement are usually considered to not be of consequence in the regions of soil extending beyond the ten percent gradient line of the pressure bulb. The assumed extent of this gradient, in terms of width and depth, is given in Figure 4-4.

Footing Overlap

An examination of Figure 4-4 will reveal 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. An obvious solution to this problem is to combine the two footings into one, as illustrated in Figure 5-8.

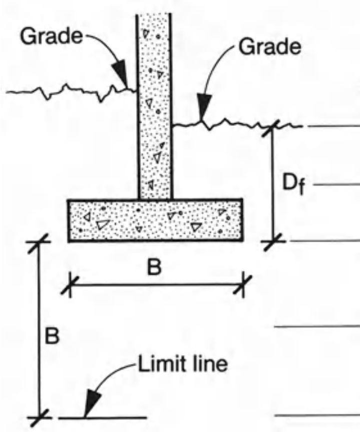
4-6. EFFECT OF GROUND WATER ON ULTIMATE BEARING CAPACITY

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 4-5.

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, the submerged weight of the soil is:

$$\gamma - 62.4 \quad \text{(pcf)}$$



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	—	weighted average
at ⑤	—	normal weight

FIGURE 4-5. The reduction in unit weight of a soil, due to ground water.

4-7. GENERAL CONSIDERATIONS OF SETTLEMENT

Settlement is a term used to describe the action whereby a footing will subside 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 will be 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, but it can be controlled by limiting the allowable soil bearing pressure for which the footing may be designed.

In the interest of brevity and with very little loss of accuracy it is recommended that settlement in the soil below the one-tenth point in the pressure bulb be ignored. 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 completely different characteristics. The settlement attributable to each of these layers must first be computed individually and then all the settlements must be added together to determine the overall effect.

The size of the pressure bulb and the depth to the one-tenth point varies with the size of the footing. 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 4-6.

The pressure bulb of the smaller footing is solely contained within the layer of dense soil and its settlement will depend solely on the response of that soil to load. The pressure bulb of the larger footing extends through the dense soil down into the layer of weaker soil. Its settlement, therefore, is a function of the cumulative

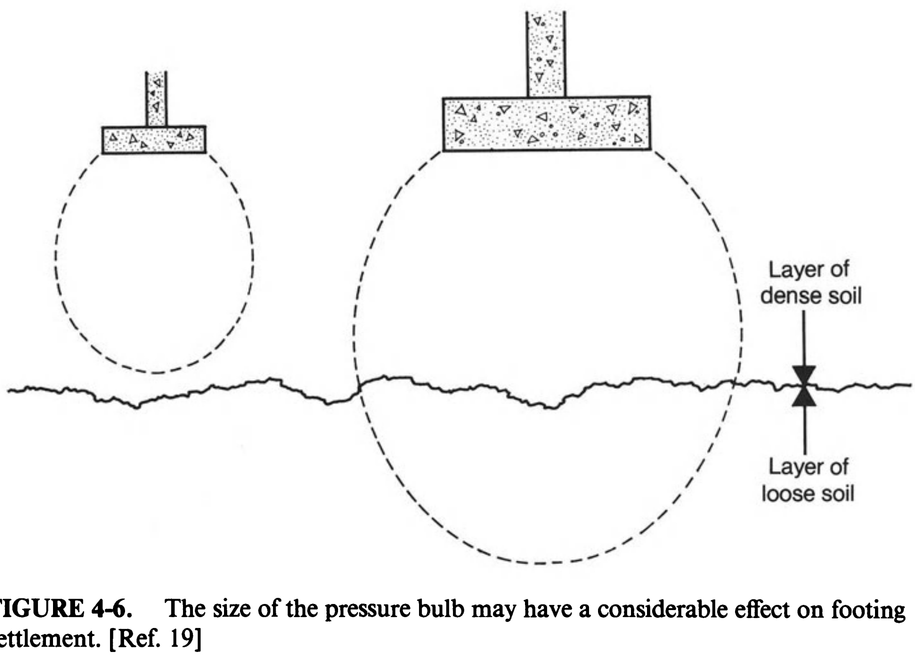


FIGURE 4-6. The size of the pressure bulb may have a considerable effect on footing settlement. [Ref. 19]

response of both layers. Computations for settlement of the larger footing, therefore, should be made for each layer individually, and then added together. It is usually considered sufficient to compute settlements for only those layers of soil which occur within the depths specified in Figure 4-5.

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, and 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 4-7.

An examination of the arrangement of particles in Figure 4-7 will show that even though there is particle to particle contact throughout the mass, the arrangement is not stable. When subjected to a superimposed load, the particles will shift and rearrange themselves until a relatively solid mass is formed. This action is called *intergranular slippage.* The extent of this slippage, and therefore the amount of settlement, depends upon the variation of particle size, 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.

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

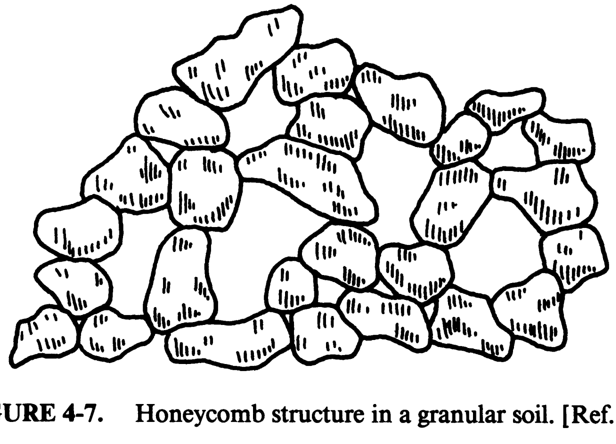


FIGURE 4-7. Honeycomb structure in a granular soil. [Ref. 10]

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 downward load the balloon, and the soil, 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 in-situ 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, representative 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 particle deformation is insignificant compared to that caused by the other two actions and may be safely 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 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; they are mostly filled 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. For a discussion on the interaction between clay and water refer to Chapter 11.

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 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 4-2.

TABLE 4-2. Rate of Settlement[a]—Sand and Clay.

	During Construction	During Service
Buildings built on sand	90%	10%
Buildings built on clay	10%	90%

[a] Percentages are for general guidelines only.

Permissible Settlement

No building can be designed to eliminate settlement completely, unless their foundations bear directly on solid rock. Settlement will occur in all other buildings regardless of the type of foundation and regardless 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 which 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 settlement of all individual footings should be limited to one inch.
2. The differential settlement between any two adjacent footings should be limited to 1/360th of the horizontal distance between the footings, measured in inches.

4-8. SETTLEMENT CALCULATIONS—FOOTINGS ON SAND

It has been 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 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. The majority of the weight, therefore, will 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 an in-situ 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 classification, density, blow count, and angle of internal friction of a coarse grained soil. For classification refer to Table 1-1. For correlation refer to Tables 2-2, 2-3, and 3-2 and Figure 2-4.

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:

a. Pebbles or small pieces of stone can impede the driving of the casing or the sampler spoon.
b. 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 that indicated by the angle of internal friction of the sample taken at that depth. The approximate correlation between these two properties is given in Figure 2-4.

It has been the experience of the author that the settlement of a footing on granular soil can be kept within acceptable limits by placing an upper limit on the allowable soil bearing pressure determined by the equations given in Section 4-3. The recommended limit, whose value is given below, is based on the blow count N occurring directly beneath the footing:

$$q_a = 0.10N\text{ tsf} = 200N\text{ psf} \tag{4-6}$$

When the allowable soil bearing pressure is limited by blow count, the total settlement of a typical footing will not exceed one inch. Differential settlement between adjacent footings will, of course, be less. Prior to applying Formula (4-6), however, it may be necessary to modify the blow count by certain correction factors, as described in the paragraphs which follow.

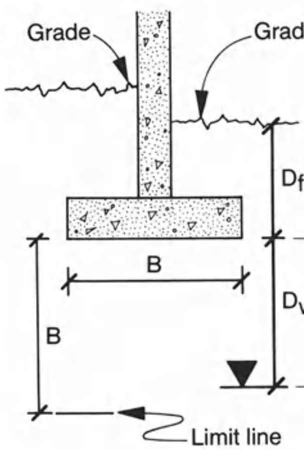
Modification in Blow Count Due to Ground Water

For equal loading, a footing bearing on saturated sand will settle more than a footing on dry sand. Subject to the level at which ground water occurs, the N values taken from the boring log should be reduced by correction factor C_w. Only those blow counts which occur between the water table and an arbitrarily chosen limit line located a distance beneath the footing equal to the width of the footing should be reduced. This condition, and the modifications required by it, are indicated in Figure 4-8.

Modification in Blow Count Due to Release of Overburden

Blow counts quantitatively indicate the density of the sand existing 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 substantial excavation may be 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. The blow count used for design, therefore, should be reduced by multiplying the original blow count from the borings by the correction factor C_n, whose value is given in Figure 4-9.

As the density of the sand is decreased when overburden is removed, it is also



Water level	Type of modification	Correction factor (C_W)
at 1	reduce N values	0.50
in 2	reduce N values	0.50
at 3	reduce N values	0.50
in 4	interpolate	$\frac{D_W}{2B} + 0.5$
at 5	do not reduce	1.00

FIGURE 4-8. Blow Count Modification Factor C_w, as due to ground water.

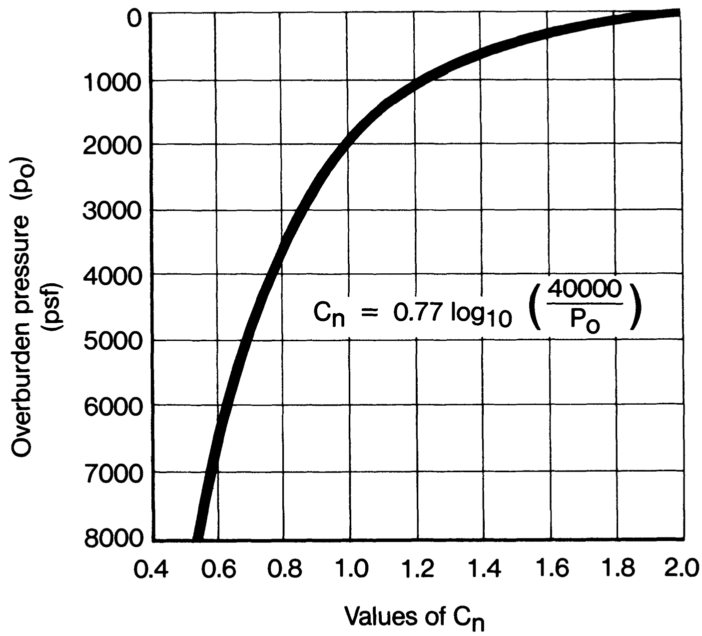


FIGURE 4-9. Blow Count Modification Factor C_n, as due to the release of overburden. [Ref. 16]

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.

4-9. RECOMMENDATIONS FOR DESIGN OF FOOTINGS ON SAND

The allowable soil bearing pressure must satisfy two different criteria, (a) soil failure, and (b) excessive settlement. It is recommended that both be considered separately and that the lesser shall govern.

Unless design charts, such as those given in Figure 4-12 are available, the following procedure may be used to determine the allowable soil bearing pressure for which a footing may be designed:

1. Compute the ultimate soil bearing capacity q_d, using Formulas (4-1) through (4-4) as applicable.
2. Use the value of q_d from step 1 to compute the allowable soil bearing pressure q_a, as given by Formula (4-5).

3. Compute the recommended limiting value of q_a based on settlement, and as given in Formula (4-6).
4. Use the lesser value of q_a, as determined by Steps 2 and 3.

The allowable soil bearing pressure can be expressed in the form of a series of design charts. The development of these charts must consider both soil failure and excessive settlement. Soil failure will govern footings of relatively narrow width, settlement will govern all others. This leads to the concept of *critical footing width*, as illustrated in Figure 4-10.

The two curves in the figure intersect at a point which is called the critical width. It can be readily seen that soil failure governs design for all footing widths less than the critical width, while settlement governs design for all footing widths larger than the critical width. The numerical value of the critical width can be determined by equating the applicable formulas for soil failure and settlement. Curves providing numerical values for the allowable soil bearing pressure as a function of footing width can then be drawn and used for design. The following computations illustrate the procedure used in the development of these curves. Because square footings are by far the more frequently used of all different footing shapes, this procedure is directed only toward development of design charts for square footings.

$$\begin{matrix} q_a \text{ based on soil failure} \\ \text{from Formula (4-5)} \end{matrix} = \begin{matrix} q_a \text{ based on settlement} \\ \text{from Formula (4-6)} \end{matrix} \qquad (4\text{-}7)$$

For a square footing bearing on sand:

$$\frac{(\gamma D_f N_q + 0.4B\gamma N_\gamma) - \gamma D_f}{SF} = 200N \quad (\text{Note: c} = 0)$$

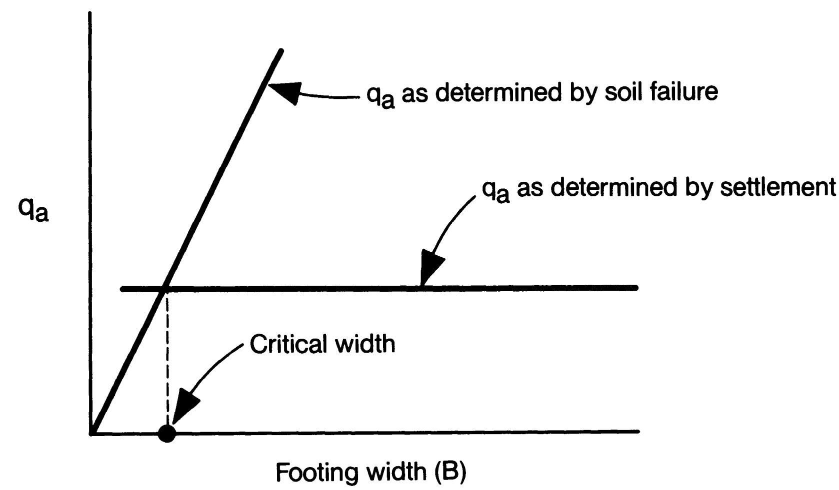


FIGURE 4-10. Critical footing width, where the allowable soil bearing pressure as determined by soil failure and settlement are equal.

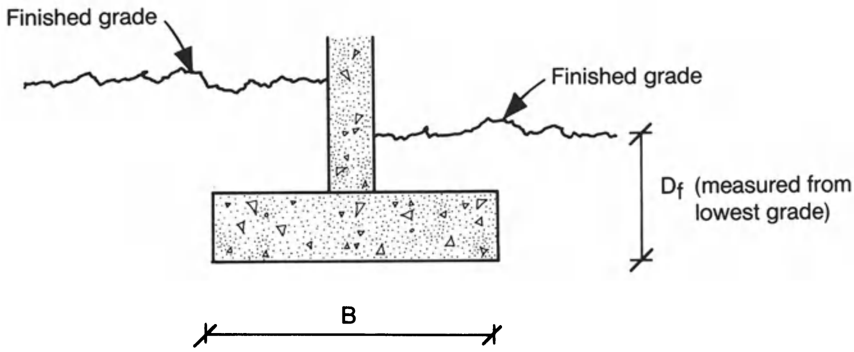


FIGURE 4-11. Definition of the parameter D_f/B, as used in Formula (4-8).

And by rearranging this equation:

$$\frac{D_f}{B}(N_q - 1) + 0.4N_\gamma = \frac{200N \times SF}{\gamma B} \tag{4-8}$$

This is the general equation from which the critical width of a square footing can be computed, provided that the blow count, soil density, and parameter D_f/B are known. The definition of this parameter is given in Figure 4-11.

The charts of Figure 4-12 are the end result of using the aforementioned procedure, and are based on an assumed soil density of 110 pcf. The number in parentheses following the blow count is the critical width of the footing as defined by Formula (4-8). The charts have a built-in safety factor of 3 against soil failure. The settlement of an individual footing should normally be expected to not exceed one inch. Differential settlement between adjacent footings, of course, will be less.

N values must be modified to account for any effects of ground water and release of overburden before entering the charts.

As an example of the use of this procedure, the width given in Figure 4-12 corresponding to $N = 10$ and $D_f/B = 0.5$ will be verified. For $N = 10$:

1. From Figure 2-4, find $\phi = 30°$.
2. Compute $N_q = 18.4$ and $N = 15.1$, using the formulas of Section 4-3.
3. Then, substituting into Formula (4-8):

$$0.5\,(18.4 - 1) + 0.4 \times 15.1 = \frac{200 \times 10 \times 3}{110B}$$

From which $B = 3.7$ feet.

Although developed for square footings, these charts can conservatively be used for continuous, round, and rectangular footings without modification. For round footings use an equivalent square footing having the same area, and for a rectangular footing use the least width.

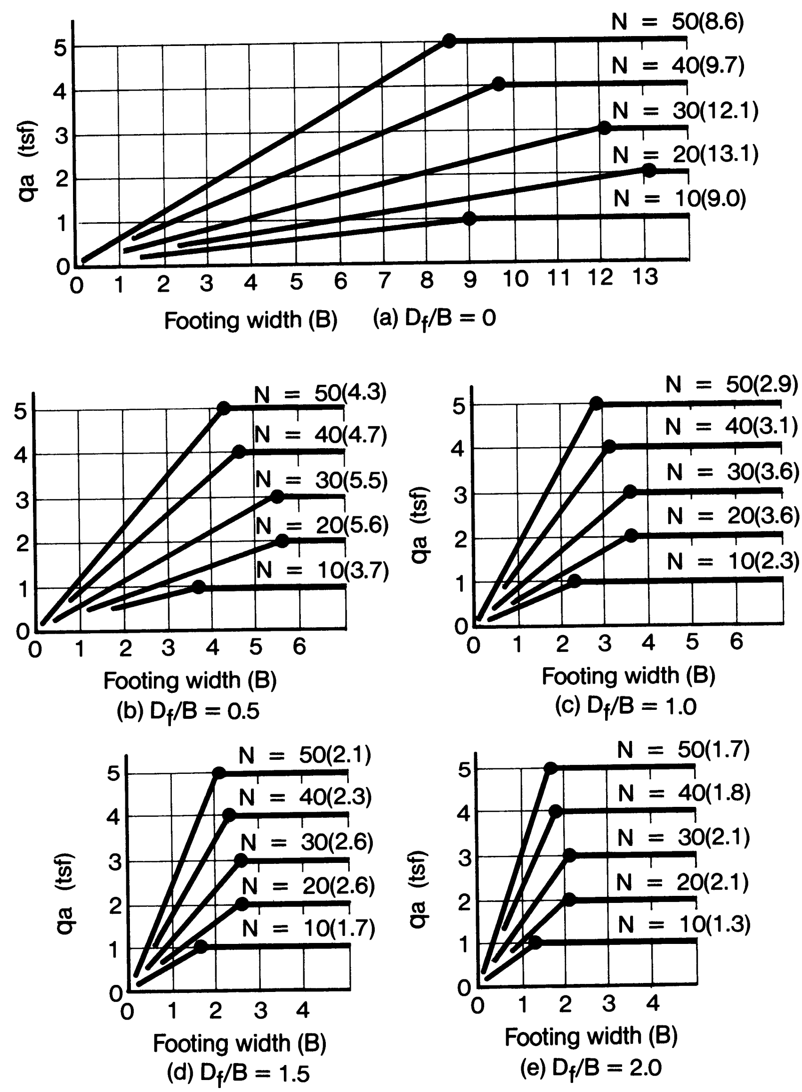


FIGURE 4-12. Design charts for footings bearing on sand.

It is recognized that these charts are somewhat conservative in regards to settlement. It is noted that the cost differential between two ton soil and two and one-half ton soil is a very small percentage of the cost of the project. The return in additional insurance against adverse settlement is usually considered to be well worth the extra cost.

Footings should never be placed directly at grade, the possibility of which is suggested by Figure 4-12(a). This figure is included only to provide information relative to the upper limit of critical width.

4-10. THE THEORY OF SETTLEMENT— FOOTINGS ON CLAY

It has been noted that the settlement of a footing on clay is primarily a function of the reduction in void ratio. The magnitude of this reduction depends primarily upon the three following things:

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

A discussion of the past history of loading requires the introduction of three new terms, *overburden, normally loaded soils,* and *preloaded soils.*

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 which 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, which 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. 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 4-3.

Reduction in Void Ratio as a Function of Pressure

A soil subjected to an increase in pressure will experience a reduction in void ratio. This reduction in turn, will lead to settlement. The concept of reduction in void ratio as a function of pressure is illustrated in Figure 4-13.

Settlement is linear, rather than volumetric. In order for the concept of settle-

TABLE 4-3. 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*	much less than *LL*

[a] A comparison of the difference between water content and liquid limit can usually be used to determine whether the soil is normally loaded or preloaded.
For information relative to liquid limit, refer to Section 11-6.

ment to be truly linear, as described herein, it must be assumed that the soil under discussion is restrained against lateral deformation.

In this figure, and in the calculations which follow, the subscript *o* refers to the original geometry of the soil. The subscript *f* refers to the final geometry of the soil after it has been subjected to an increase in loading.

The void ratios of the soil before and after being subjected to an increase in load, are:

$$e_o = \frac{V_{vo}}{V_s} \quad \text{and} \quad e_f = \frac{V_{vf}}{V_s}$$

When the volume of solids is taken as unity:

$$V_{vo} - V_{vf} = e_o - e_f$$

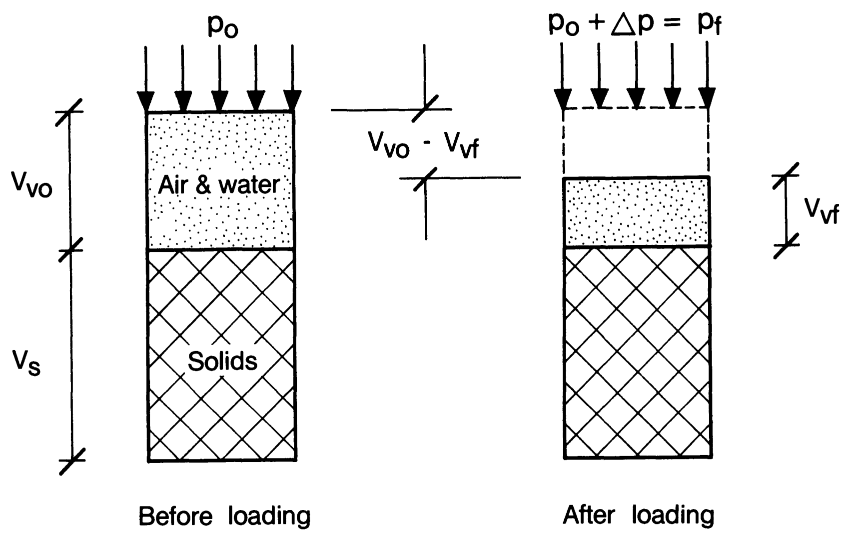


FIGURE 4-13. The concept of settlement as a function of reduction in void ratio.

Therefore an increase in pressure $p_f - p_o$ causes a corresponding decrease in void ratio $e_o - e_f$.

Settlement as a Function of Reduction in Void Ratio

Linear strain, by definition, is the ratio of the reduction in length to the original length, therefore:

$$\text{linear strain} = \frac{e_o - e_f}{1 + e_o}$$

If linear strain is assumed to be relatively constant throughout a clay layer having a thickness H, then the total settlement ΔH in that layer is:

$$\text{settlement } \Delta H = \left[\frac{e_o - e_f}{1 + e_o}\right] H$$

This formula may be used to compute the anticipated amount of settlement, provided that the reduction of void ratio in the clay layer is known. The relationship between void ratio and pressure can be established in the laboratory for any given layer of clay by running a consolidation test on an undisturbed sample of the soil.

4-11. 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. Consolidation, therefore, is always accompanied by a reduction in volume, and correspondingly, a reduction in void ratio.

A consolidation test is a specialized test which is 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

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. An example of the type of apparatus used in this test is illustrated in Figure 4-14.

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

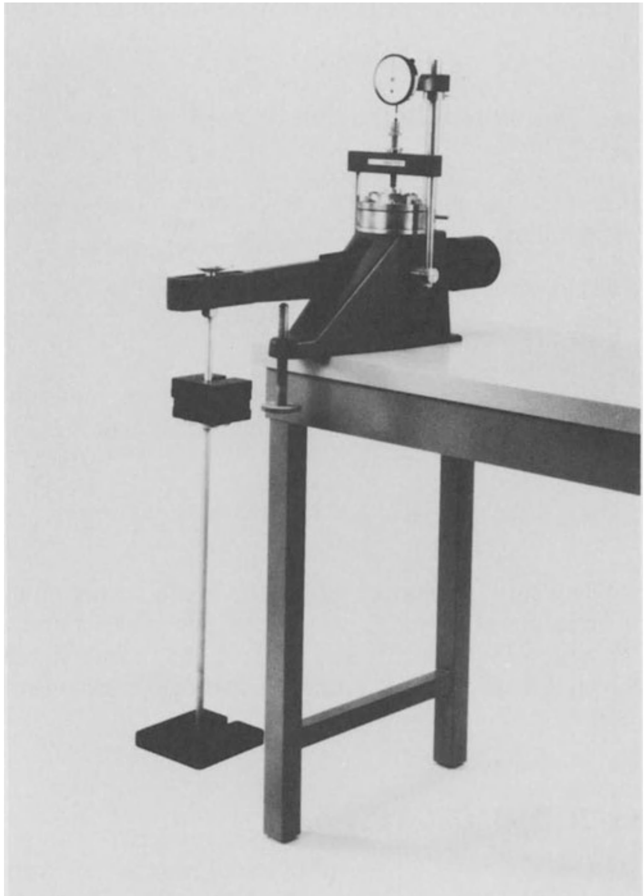


FIGURE 4-14. Consolidation test apparatus, as used to determine the variation in void ratio as a function of pressure and time. [Ref. 17]

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. Additional information, such as soil density, moisture content, liquid limit, plastic limit, and the specific gravity of the solid constituents will also be determined.

To perform this test an axial load is applied 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. This load is maintained at a constant level until this time-related reduction in height has, for all intents and purposes, ceased. The test is then repeated several times, but each time with a greater load, usually doubled. Customary 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 gage which records the deformation is then reset before the next load is applied to the sample. It must be noted, therefore, 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 linear deformation as a function of time. One such set of curves is shown in Figure 4-15.

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 consolidation.* Of the two, primary consolidation is by far the more critical when considering the settlement of a building. It is for this reason that

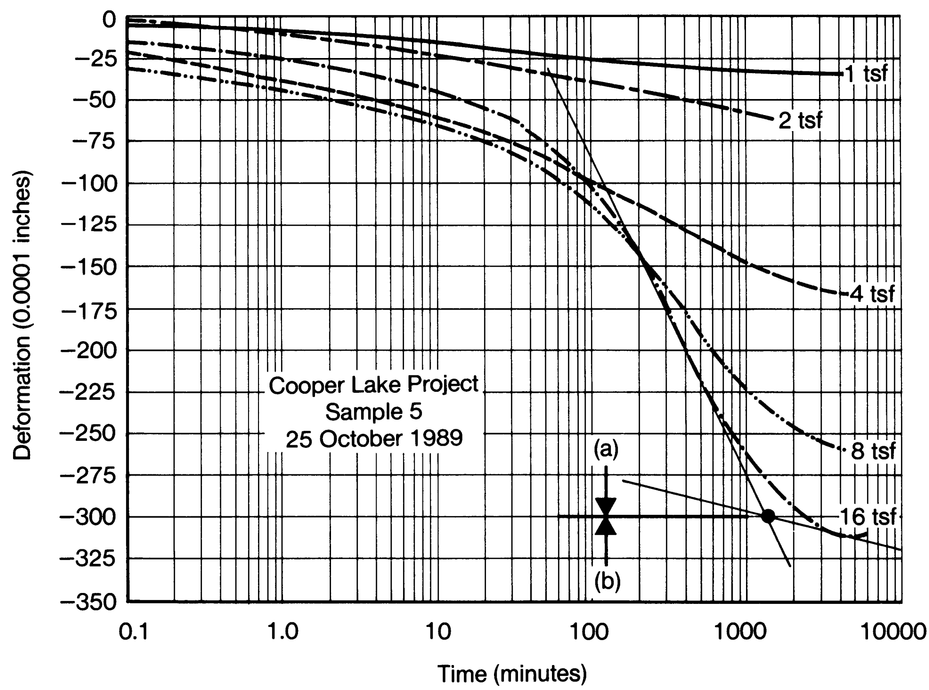


FIGURE 4-15. The results of a particular consolidation test, in which deformation was determined as a function of time. (a) indicates the range of primary consolidation, and (b) indicates the range of secondary consolidation. [Ref. 22]

ASTM D-2435 includes a procedure whereby the transition from primary consolidation to secondary consolidation can be identified in terms of deformation. This deformation is determined graphically as the intersection of two straight lines drawn on the deformation-time curve. The first line is drawn through the points representing the final readings and exhibiting 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 in Figure 4-15.

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 previously determined during the test procedure. These values can be used to determine the volume of solids by rearranging Formula (2-8):

$$V_s = \frac{W_s}{62.4G_s}$$

The volume of voids is then determined:

$$V_v = 1 - V_s$$

The void ratio existing at the start of the test is computed from Formula (2-3):

$$e_o = \frac{V_v}{V_s}$$

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.

$$e = \frac{V_v}{V_s} = \frac{h_v}{h_s}$$

It is noted that equivalent height of solids in the sample can be determined multiplying the volume of solids in one cubic foot by the initial height of the sample.

The void ratios corresponding to the theoretical point of 100% compaction are then computed for the various test pressures and plotted on an "*e*-log *p*" curve, a sample of which is shown in Figure 4-16.

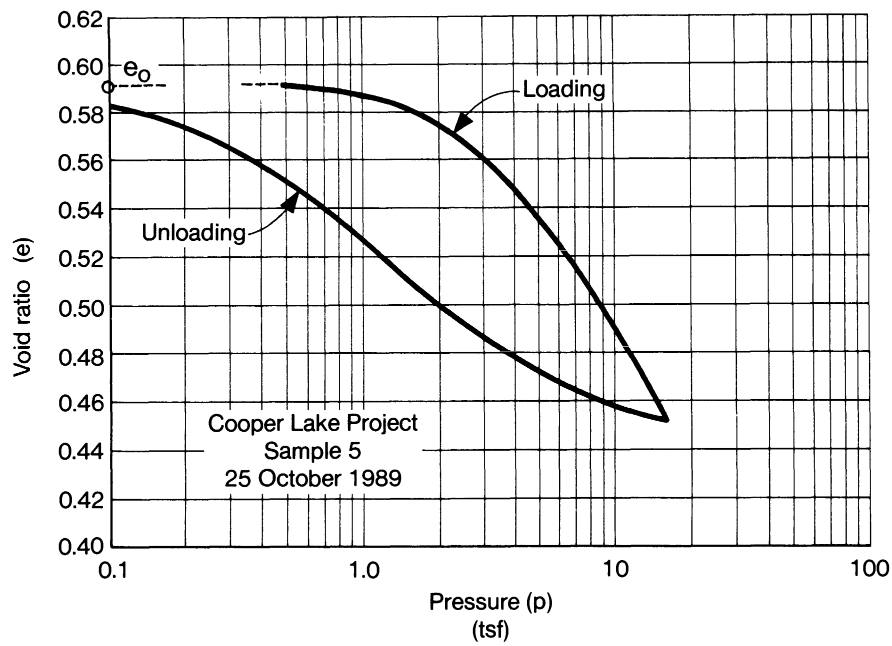


FIGURE 4-16. This curve, which is a continuation of the consolidation test of Figure 4-15, expresses the variation in void ratio as a function of pressure. This is commonly called an *e*-log *p* curve. [Ref. 22]

It must be noted that the consolidation test as described herein is both costly and time consuming. A single operation may take up to twenty-four 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 some assurance of reasonable accuracy.

4-12. SETTLEMENT CALCULATIONS—FOOTINGS ON CLAY

The Formula for Settlement

Calculations of settlement are based on the concept presented in Section 4-10, in which the formula for settlement, as repeated herein, was derived:

$$\Delta H = \left[\frac{e_o - e_f}{1 + e_o}\right] H \qquad (4\text{-}9)$$

A restatement of the variables used in the settlement calculations is as follows:

e_o is the void ratio of the in-situ soil prior to construction, and corresponds to the pressure p_o, measured at the same depth.

e_f is the void ratio of the soil after construction has been completed, and corresponds to the pressure p_f, measured at the same depth. The numerical value of e_f is taken from the e-log p curve.

p_o is the in-situ overburden pressure existing prior to the start of construction. This pressure is equal to the weight of the earth above the plane, and should also theoretically include the prorated weight of any buildings on the site. Since the pressure caused by the weight of these buildings is sharply reduced by the pressure bulb effect, their weight is usually ignored. When the water table is above the plane being considered, the submerged weight must be used for all earth located above the plane but below the table.

Δp is 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 was designed.

p_f is the total pressure on the plane for which settlement must be considered. p_f is numerically equal to $p_o + \Delta p$.

It should be noted that natural soil deposits are frequently layered, and the properties of one layer may differ considerably from those of another. Settlements should be computed for each of the individual layers and then summed up. The void ratios and pressures required for each computation are those assumed to exist on a plane located at the mid-height of the layer of soil whose settlement is being computed.

It should also be recalled that settlements in soils beneath the 10% gradient line are generally considered to not be of consequence.

Simplified Calculations for Δp

The numerical value of the increase in pressure, Δp, may be found by a procedure in which it is assumed that the superimposed load distributes into the ground on a gradient whose shape is that of the frustum of a right rectangular pyramid. The sides of the pyramid are assumed to slope one unit horizontal for each two units vertical. Although only an approximation, this procedure is frequently used by architects and engineers because of its relative simplicity and because it is on the conservative side. This procedure is illustrated in Figure 4-17, in which:

Q is the load on the footing, pounds

B_1 and B_2 are the length and width of the footing, feet

y is the depth below the footing to the mid-height of the layer under consideration, feet

Δp is the pressure induced on the plane of interest by the force Q, psf

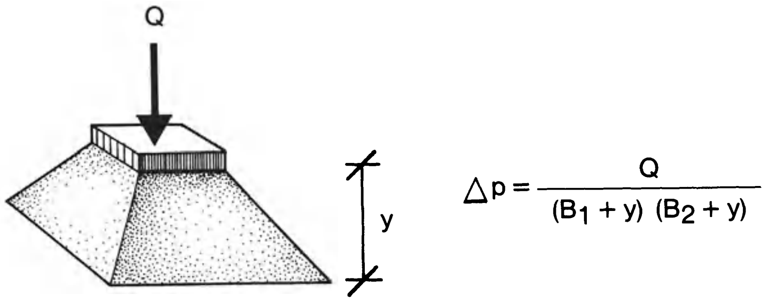


FIGURE 4-17. A conservative approximation for determination of the increase in pressure Δp, at a particular depth below the footing.

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, in-place density, moisture content, water table, and a visual description of the soil can be obtained from the test borings. Other information, such as dry density, liquid and plastic limits, specific gravity of the solids and the in-place void ratio can be obtained from the laboratory analysis. Then:

1. Compute e_o, p_o, Δp, and p_f.
2. Read e_f off the e-log p curve.
3. Compute settlement for the layer of soil under consideration, using Formula (4-9).
4. Repeat the above steps for all other layers of soil which occur within the 10% gradient zone.
5. Obtain the total settlement by adding the individual settlements.

The Effect of Site Excavation

It is emphasized that settlement is caused by a reduction in void ratio, which in turn is caused by an increase in pressure. What is the effect, however, when large areas of the site are excavated, thereby removing all, or at least part of the overburden pressure? The answer is that the soil will then rebound, and as the soil rebounds, its void ratio will increase. This condition is illustrated in Figure 4-18, where points shown as filled circles indicate the condition that would exist without site excavation. Points shown as open circles indicate the condition existing after site excavation.

The effect of removing a part or all of the overburden pressure is to shift the p_o point to the left on the e-log p curve. Since Δp does not change, the p_f point will shift an equal amount. The slope of the e-log p curve is generally steeper in the region where the pressure is greater. It would be expected, then, that for an equal

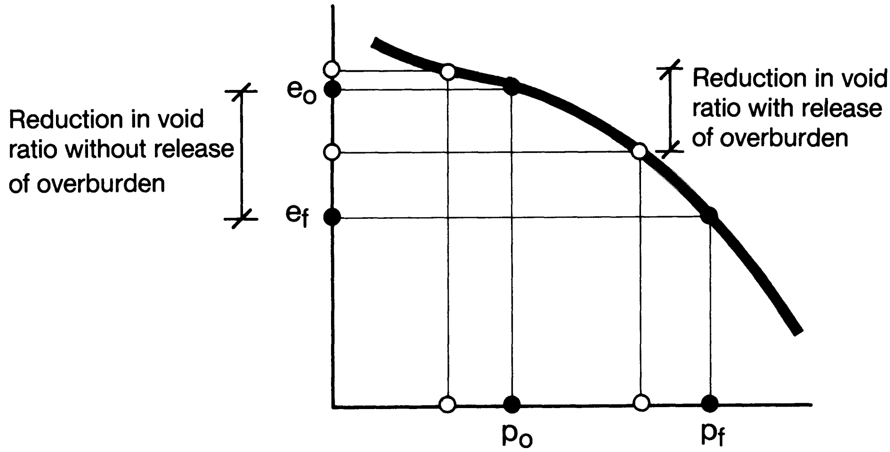


FIGURE 4-18. The effect of site excavation on void ratio.

reduction in pressure, the corresponding reduction in void ratio would be greater at p_f than at p_o.

The designer is reminded that settlement on soils of clay is a long term condition, and that there are many intangibles between theory and practice. For these reasons the author is convinced that a conservative approach to this problem is in the best interest of the client.

4-13. RECOMMENDATIONS FOR DESIGN OF FOOTINGS ON CLAY

It would be desirable to correlate the settlement of clay to blow count, as was done in the case of footings on sand. Blow count, it has been noted, provides a reasonably reliable measure of the resistance of sand to settlement. In the case of clay, however, blow count has been found to be not so reliable. The footing, therefore, should be sized tentatively on the basis of soil failure, using the allowable soil bearing pressure as specified in Section 4-4.

Representative footings should then be selected for computation of settlement in accordance with the procedures outlined in Section 4-12. In most instances the computed settlement of footings sized on the basis of soil failure will be within acceptable limits. For those instances when the computed settlement is considered to be excessive, the allowable soil bearing pressure should be reduced. Subsequent design can usually be based on this reduced soil bearing pressure without the need to make continuing calculations on settlement.

The computations for the allowable soil bearing pressure of a footing on clay can be somewhat simplified, as demonstrated herein.

For a continuous footing Formulas (4-1) and (4-5) give:

$$q_a = \frac{1.0cN_c + \gamma D_f N_q - \gamma D_f}{SF} \quad \text{(Note: } N_\gamma = 0\text{)}$$

For a pure clay $\phi = 0$, $N_c = 5.14$, $N_q = 1.0$, and $c = 0.5q_u$; therefore:

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

And for all practical purposes:

$$q_a = 0.86q_u \quad \text{for a continuous footing} \quad (4\text{-}10)$$

$$q_a = 1.02q_u \quad \text{for a square footing} \quad (4\text{-}11)$$

$$q_a = 1.02q_u \quad \text{for a round footing} \quad (4\text{-}12)$$

$$q_a = 0.86q_u a_1 \quad \text{for a rectangular footing} \quad (4\text{-}13)$$

4-14. RECOMMENDATIONS FOR DESIGN OF FOOTINGS ON MIXED GRAINED SOIL

A mixed grained soil, by definition, includes both coarse and fine grained soil fractions. The characteristics of this type of soil, however, are very likely to be dominated by one or the other of those soil fractions. In many cases the dominant fraction can be identified by visual examination and by physical manipulation of the soil. In other cases laboratory tests must be used to properly identify the soil, and to determine its characteristics. The more obvious characteristics of each kind of soil are listed below:

Pure clay is inherently a cohesive material, and for that reason can readily be molded. It also possesses a measurable unconfined compression strength, but has no measurable angle of internal friction.

Pure sand is completely cohesionless and cannot be molded at all, nor can it be tested for unconfined compression strength. Pure sand does possess a measurable angle of internal friction.

When it can be established that a particular mixed grained soil exhibits the predominant characteristics of either a sand or a clay, then the analysis of that soil should be made on the basis of the dominant fraction, as described elsewhere in this chapter.

When the mixed grained soil exhibits the general characteristics of both sand and clay then the soil should be designed as a mixed grained soil. In this case the ultimate soil bearing capacity should be determined by Formulas (4-1) through (4-4), as applicable. Settlement should be computed using the method shown in Section 4-12, as described for a footing bearing on clay.

It should be noted that design on the basis of a mixed grained soil should only be made when it is evident that the soil actually exhibits some of the characteristics of both a granular and a cohesive soil. This should include a measurable cohesion and angle of internal friction. When in doubt, it is much safer and much more prudent to treat the soil as either purely granular, or purely cohesive. This will give the lesser value in the ultimate bearing capacity formula.

4-15. CLOSING RECOMMENDATIONS

General

The true behavior of soils under load can rarely be predicted. There are many intangibles, not the least of which are the differences in the perception of this behavior by different architects and engineers. 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 the foundations. The designer should recognize these problems, and wherever possible should direct his design and construction schedule to give the contractor every opportunity to utilize 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 that area to determine what construction techniques are suitable for the site under consideration. The advice and counsel of a knowledgeable contractor can prove invaluable to the success of the project when he is consulted at an early stage of the design process.

Minimum Soil Pressure

It is the opinion of the author that soils whose allowable bearing pressure is one tsf or less should never be used for the support of any kind of building foundation, and that pressures less than one and one-half tsf should be highly suspect. These low pressures invite problems with settlement. The pressures referred to here are the allowable pressures obtained after applying a safety factor to the computed ultimate bearing pressure.

Local Shear

Footings founded on loose sand, or soft or sensitive clay, may fail at a lesser bearing pressure than that which is 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 and Whitman (Ref. 11).

Support Options

When the subgrade consists of satisfactory bearing material except for a relatively thin layer at the surface, then 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, thereby reducing the imposed soil pressure considerably. Particular reference is made to the effect that this large bulb pressure will have on settlement, as illustrated in Figure 4-6.

When the subgrade below the normal footing elevation is simply inadequate, and when this condition extends for some depth, then the only alternative is to extend the loads down through the inadequate soil and transfer them directly to acceptable bearing. This requires the use of a pile or pier foundation, as described in Chapter 6.

Minimum Footing Width

It has been the experience of the author that from the practical standpoint of digging a trench, maintaining the banks, and placing reinforcement and concrete, the minimum width of any footing should reasonably be set at two feet. Also, where possible, footings should be cast in earth forms. For those instances in which the banks collapse into the excavation, wood edge forms must be used, as described in Section 5-2. These forms, however, must be removed after the concrete has hardened. The spalled off area of earth adjacent to the footing must be backfilled and compacted to the density specified in one of the following sections of this book:

1. For coarse grained soils, relative density, Section 10-6
2. For fine grained soils, modified proctor density, Section 10-7

4-16. 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 projects of limited size; 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. Nor do these values usually consider variation of soil with depth, size of footing, ground water or permissible settlement. It is the opinion of the author that the only proper way to establish safe bearing values is to conduct a thorough testing procedure, as discussed in various areas of this text.

TABLE 4-4. Presumptive Bearing Pressures.

Soil Description	Bearing Pressure, tsf
Quick clays	not acceptable
Very soft, soft or playa 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

Presumptive bearing pressures do serve a useful purpose, however, in that they act as a generalized guideline of what kind of pressures can generally be expected of a particular soil. Table 4-4 has been compiled by the author from different codes and from his own experience.

4-17. SAMPLE PROBLEMS

Example 4-1

Required: To determine the allowable soil bearing pressure beneath a 2′-6″ wide continuous footing located 3′-6″ below the ground surface.
Given: The soil is mixed grained. Laboratory tests have found:

$$\phi = 28^\circ \qquad \gamma = 118 \text{ pcf} \qquad q_u = 0.2 \text{ tsf} = 400 \text{ psf}$$
$$\text{cohesion } c = 0.5\, q_u = 200 \text{ psf}$$

Bearing capacity factors from formulas:

$$N_c = 25.8 \qquad N_q = 14.7 \qquad N_\gamma = 10.9$$

From Formula (4-1),

$$\begin{aligned} q_d = {} & 1.0 \times 200 \times 25.8 + 118 \times 3.5 \times 14.7 \\ & + 0.5 \times 2.5 \times 118 \times 10.9 = 12{,}839 \text{ psf} \end{aligned}$$

and

$$q_a = \frac{12839 - 118 \times 3.5}{3} = 4142 \text{ psf}$$

Example 4-2

Required: To determine the allowable soil bearing pressure beneath a 5′-0″ square footing located 6′-0″ below the surface of the ground.
Given: The soil is identified as mixed grained. Laboratory tests find:

$$\phi = 22^\circ \qquad \gamma = 115 \text{ pcf} \qquad q_u = 0.8 \text{ tsf} = 1600 \text{ psf}$$
$$\text{cohesion } c = 0.5\, q_u = 800 \text{ psf}$$

Bearing capacity factors from formulas:

$$N_c = 16.9 \qquad N_q = 7.8 \qquad N_\gamma = 4.1$$

From Formula (4-2):

$$q_d = 1.2 \times 800 \times 16.9 + 115 \times 6.0 \times 7.8 \\ + 0.4 \times 5.0 \times 115 \times 4.1 = 22{,}549 \text{ psf}$$

and

$$q_a = \frac{22549 - 115 \times 6.0}{3} = 7286 \text{ psf}$$

Example 4-3

Required: To determine the allowable soil bearing pressure beneath a 12′-0″ diameter water tank whose footing is located 6′-6″ below the surface of the ground.
Given: The soil is identified as mixed grained. Laboratory tests find:

$$\phi = 18^\circ \qquad \gamma = 120 \text{ pcf} \qquad q_u = 1.2 \text{ tsf} = 2400 \text{ psf}$$
$$\text{cohesion } c = 0.5\ q_u = 1200 \text{ psf}$$

Bearing capacity factors from formulas:

$$N_c = 13.1 \qquad N_q = 5.3 \qquad N_\gamma = 2.1$$

From Formula (4-3):

$$q_d = 1.2 \times 1200 \times 13.1 + 120 \times 6.5 \times 5.3 \\ + 0.6 \times 6.0 \times 120 \times 2.1 = 23{,}905 \text{ psf}$$

and

$$q_a = \frac{23905 - 120 \times 6.5}{3} = 7708 \text{ psf}$$

Example 4-4

Required: To determine the allowable soil bearing pressure beneath a 4′-0″ × 12′-0″ rectangular footing located 4′-6″ below the surface of the ground.
Given: The soil is mixed grained. Laboratory tests find:

$$\phi = 26^\circ \qquad \gamma = 116 \text{ pcf} \qquad q_u = 0.4 \text{ tsf} = 800 \text{ psf}$$
$$\text{cohesion } c = 0.5\ q_u = 400 \text{ psf}$$

Bearing capacity factors from formulas:

$$N_c = 22.3 \qquad N_q = 11.9 \qquad N_\gamma = 7.9$$

From Table 4-1:

$$a_1 = 1.07 \qquad a_2 = 0.46$$

From Formula (4-4):

$$\begin{aligned} q_d &= 1.07 \times 400 \times 22.3 + 116 \times 4.5 \times 11.9 \\ &\quad + 0.46 \times 4.0 \times 116 \times 7.9 = 17{,}442 \text{ psf} \end{aligned}$$

and

$$q_a = \frac{17442 - 116 \times 4.5}{3} = 5640 \text{ psf}$$

Example 4-5

Required: To determine the allowable soil bearing pressure beneath a 6′-0″ square footing located 8′-0″ below the surface of the ground.
Given: The soil is a grayish medium sand. Laboratory tests find:

$$\phi = 33° \qquad \gamma = 110 \text{ pcf} \qquad q_u = 0 \text{ (cohesion } c = 0)$$

Bearing capacity factors from formulas:

$$N_c = \text{—} \qquad N_q = 26.1 \qquad N_\gamma = 24.4$$

From Formula (4-2):

$$\begin{aligned} q_d &= 110 \times 8.0 \times 26.1 \\ &\quad + 0.4 \times 6.0 \times 110 \times 24.4 = 29{,}410 \text{ psf} \end{aligned}$$

and

$$q_a = \frac{29410 - 110 \times 8.0}{3} = 9510 \text{ psf}$$

Example 4-6

Required: To determine the allowable soil bearing pressure beneath a 4′-0″ by 8′-0″ rectangular footing located 3′-0″ below the surface of the ground.

Given: The soil is a medium dense sand. Test borings find: $N = 30$. Laboratory tests find:

$$\gamma = 122 \text{ pcf} \qquad q_u = 0 \qquad (\text{cohesion } c = 0)$$

From Figure 2-4: $\phi = 36°$.
From Table 4-1: $a_2 = 0.45$.
Bearing capacity factors from formulas:

$$N_c = \text{—} \qquad N_q = 37.7 \qquad N_\gamma = 40.0$$

From Formula (4-4):

$$q_d = 122 \times 3.0 \times 37.7 \\ + 0.45 \times 4.0 \times 122 \times 40.0 = 22{,}582 \text{ psf}$$

and

$$q_a = \frac{22582 - 122 \times 3.0}{3} = 7405 \text{ psf}$$

Example 4-7

Required: To determine the allowable soil bearing pressure beneath a 2′-0″ wide continuous footing located 4′-0″ below the surface of the ground.
Given: The soil is identified as a stiff clay. Laboratory tests find:

$$\phi = 0° \qquad \gamma = 122 \text{ pcf} \qquad q_u = 1.4 \text{ tsf} = 2800 \text{ psf}$$
$$\text{cohesion } c = 0.5\, q_u = 1400 \text{ psf}$$

Bearing capacity factors from formulas:

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

From Formula (4-1):

$$q_d = 1.0 \times 1400 \times 5.14 \\ + 122 \times 4.0 \times 1.0 = 7684 \text{ psf}$$

and

$$q_a = \frac{7684 - 122 \times 4.0}{3} = 2399 \text{ psf}$$

Or, from Formula (4-10):

$$Q_a = 0.86 \times 2800 = 2408 \text{ psf}$$

Example 4-8

Required: To determine the allowable soil bearing pressure beneath a 3′-0″ × 6′-0″ rectangular footing located 7′-0″ below the surface of the ground.
Given: The soil is very stiff clay, having the following properties:

$$\phi = 0^\circ \qquad \gamma = 125 \text{ psf} \qquad q_u = 2.5 \text{ tsf} = 5000 \text{ psf}$$

$$\text{cohesion } c = 0.5\, q_u = 2500 \text{ psf}$$

Bearing capacity factors from formulas:

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

From Table 4-2: $a_1 = 1.12$.
From Formula (4-4):

$$\begin{aligned} q_d = {} & 1.12 \times 2500 \times 5.14 \\ & + 125 \times 7.0 \times 1.0 = 15{,}267 \text{ psf} \end{aligned}$$

and

$$q_a = \frac{15267 - 125 \times 7.0}{3} = 4797 \text{ psf}$$

Or, from Formula (4-13):

$$q_a = 0.86 \times 1.12 \times 5000 = 4816 \text{ psf}$$

Example 4-9

Required: To determine the allowable soil bearing pressure beneath the 12′-0″ diameter water tank of Example 4-3, assuming that ground water occurs 2′-0″ below the surface of the ground.

From Example 4-3:

$$\phi = 18^\circ \qquad \gamma = 120 \text{ pcf} \qquad q_u = 1.2 \text{ tsf} = 2400 \text{ psf}$$

$$\text{cohesion } c = 0.5\, q_u = 1200 \text{ psf}$$

$$N_c = 13.1 \qquad N_q = 5.3 \qquad N_\gamma = 2.1$$

From Figure 4-5, the water table is located in level 2. Therefore, in the $\gamma D_f N_q$ term use the weighted average, and in the $B\gamma N_\gamma$ term use the submerged weight.

$$\text{Submerged weight} = 120.0 - 62.4 = 57.6 \text{ pcf.}$$

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

From Formula (4-3):

$$q_d = 1.2 \times 1200 \times 13.1 + 76.8 \times 6.5 \times 5.3 + 0.6 \times 6.0 \times 57.6 \times 2.1 = 21{,}945 \text{ psf}$$

and

$$q_a = \frac{21945 - 76.8 \times 6.5}{3} = 7148 \text{ psf}$$

Example 4-10

Required: To determine the allowable soil bearing pressure beneath a 3′-0″ wide continuous footing located 3′-6″ below the surface of the ground.
Given: The soil is found to be a mixed grained, very loose soil. Laboratory tests find:

$$\phi = 22.5° \qquad \gamma = 105 \text{ pcf} \qquad q_u = 0.6 \text{ tsf} = 1200 \text{ psf}$$

$$\text{cohesion } c = 0.5\, q_u = 600 \text{ psf}$$

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 ⅓ before determining the bearing capacity factors, and that the numerical value of the cohesion should likewise be reduced by ⅓.

Therefore

$$\text{Effective angle of internal friction} = 22.5 \times 2/3 = 15°$$

and

$$\text{Effective cohesion} = 600 \times 2/3 = 400 \text{ psf}$$

Bearing capacity factors from formulas:

$$N_c = 11.0 \qquad N_q = 3.9 \qquad N_\gamma = 1.2$$

From Formula (4-1):

$$q_d = 400 \times 11.0 + 105 \times 3.5 \times 3.9 + 0.5 \times 3.0 \times 105 \times 1.2 = 6022 \text{ psf}$$

and

$$q_a = \frac{6022 - 105 \times 3.5}{3} = 1885 \text{ psf}$$

It is the opinion of the author that this allowable bearing pressure is so low that the premise of using spread footings for this condition is unacceptable. Refer to a discussion of local shear in Section 4-15. Alternate solutions include soil compaction, spread footings at a lower depth, or the use of piles, piers or caissons.

Example 4-11

This is the first of four problems dealing with the same example, leading ultimately to calculations of footing settlement.

Required: To determine the maximum load for which the footing can be designed, assuming the conditions as indicated in Figure 4-19.

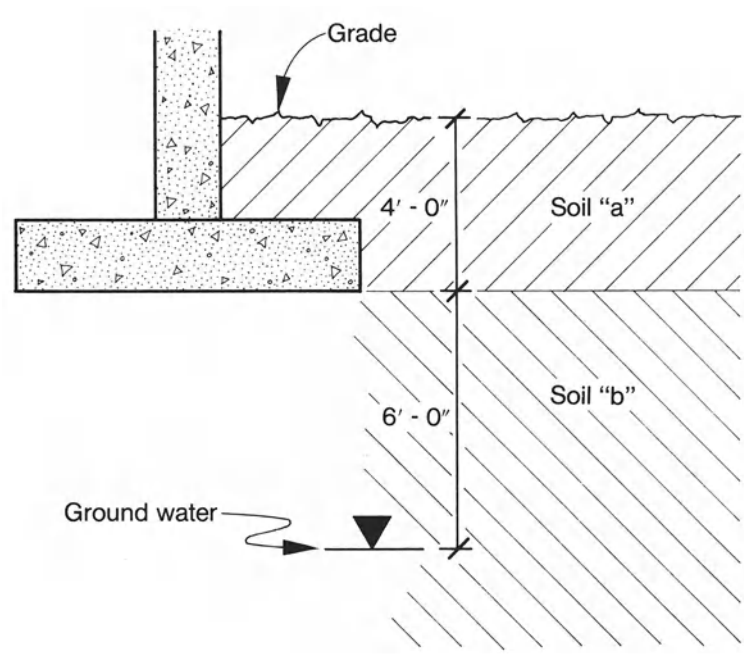


FIGURE 4-19.

The soil characteristics are as follows:

Soil "a" is a loose clay, having a density of 100 pcf.
Soil "b" is a lean, stiff clay, having a density of 127.8 pcf and an unconfined compression strength of 2.0 tsf.

Because the bearing soil is clay, $N_c = 5.14$, $N_q = 1.0$, and $N_\gamma = 0$. The cohesion is taken as $0.5\ q_u = 0.5 \times 2 \times 2000 = 2000$ psf.

From Figure 4-5, the water table is located in level 4. Therefore, in the $\gamma D_f N_q$ term, water is not a factor, and in the $B\gamma N_\gamma$ term, water would normally be a factor, but in a cohesive soil this term is deleted.

From Formula (4-2):

$$q_d = 1.2 \times 2000 \times 5.14 \\ + 100 \times 4.0 \times 1.0 = 12{,}736 \text{ psf}$$

and

$$q_a = \frac{12736 - 100 \times 4.0}{3} = 4112 \text{ psf}$$

Or, from Formula (4-11):

$$q_a = 1.02 \times 4000 = 4080 \text{ psf}$$

Maximum safe load for which footing may be designed:

$$4000 \times 8.0 \times 8.0 = 256{,}000\ \# = 256 \text{ Kips}$$

Example 4-12 (Continuing Example 4-11)

Required: To draw the e-log p curve for the lean, stiff clay.

Additional information supplied by the testing laboratory:

$$\gamma_{\text{dry}} = 106.4 \text{ pcf} \quad \text{and} \quad G_s = 2.68$$

By the procedures outlined in Section 4-11:

$$\text{Volume of solids } V_s \text{ in one cubic foot} = \frac{106.4}{62.4 \times 2.68} = 0.6362 \text{ CF}$$

$$\text{Volume of voids } V_v \text{ in one cubic foot} = 1.0000 - 0.6362 = 0.3638 \text{ CF}$$

$$\text{Void ratio } e_o \text{ at the start of the test} = \frac{0.3638}{0.6362} = 0.572$$

TABLE 4-5. Consolidation Test Data.

Load, tsf	Dial Reading at Start	Dial Reading at Finish	Consolidation 0.0001 inches	Sample Height	Void Ratio
0				0.8790	0.572
½	1	17	16	0.8774	0.569
1	8	75	67	0.8723	0.560
2	10	127	117	0.8606	0.539
4	13	231	218	0.8388	0.500
8	8	260	252	0.8136	0.455
16	0	263	263	0.7873	0.408

A consolidation test was performed on a sample of this material. The change 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 4-5.

The initial height of the sample used in the test was 0.8790 inches. Therefore, the equivalent height of solids in the sample is:

$$h_s = 0.8790 \times 0.6362 = 0.5592 \text{ inches}$$

A sample calculation of void ratio used in the *e*-log *p* curve is as follows:

$$e\,(\text{at 8 tsf}) = \frac{0.8136 - 0.5592}{0.5592} = 0.455$$

A plot of the *e*-log *p* curve is given in Figure 4-20.

Example 4-13 (Continuing Example 4-11)

Required:

1. To compute the overburden pressure existing at the mid-depth of the clay layer extending form the footing to the assumed line of the 10% pressure gradient. (Since this problem ultimately deals with settlement, this limit line is chosen in accordance with the provisions of Section 4-5.) In accordance with Figure 4-4 this gradient line is 16 feet below the footing.
2. To compute the pressure induced at this same elevation by the design load of the column footing.
3. To determine the resultant pressure p_f.

1. The overburden pressure is:

$$p_o = 100 \times 4.0 + 127.8 \times 6.0 + (127.8 - 62.4)\,2.0 = 1298 \text{ psf}$$

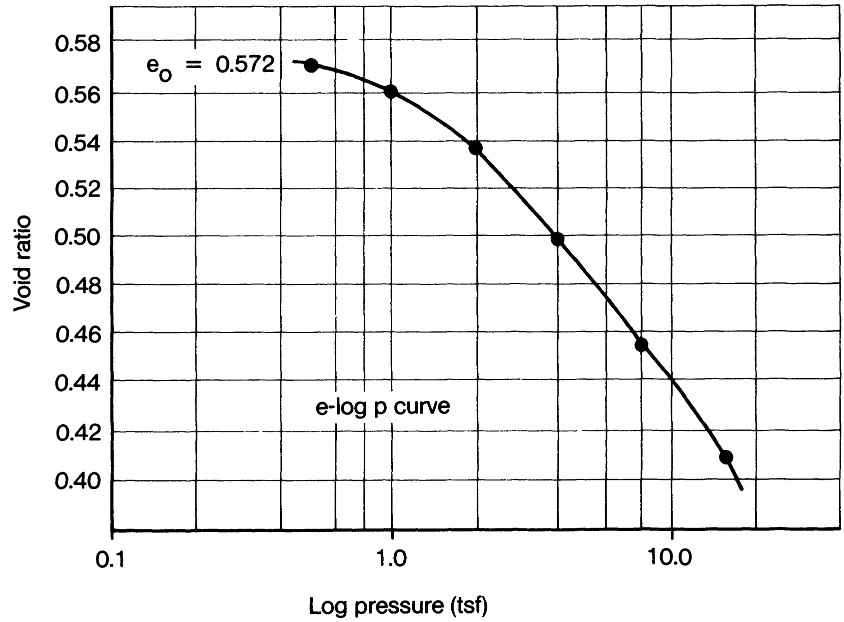


FIGURE 4-20.

2a. The induced pressure, as determined from Figure 4-3, is:

Δp = Average stress between the center and the edge of the footing, measured at the mid-depth of the 10% gradient line times the pressure intensity directly beneath the footing.

$$= \frac{0.20 + 0.32}{2} \times 4000 = 1040 \text{ psf}$$

2b. The induced pressure computed in accordance with Figure 4-17, is:

$$\Delta p = \frac{256000}{(8 + 8)(8 + 8)} = 1000 \text{ psf}$$

2c. The induced pressure is computed as being the average of the pressures induced at the corner and center of the footing.

At the corner, from Section C-4 of Appendix C,

$$n = m = 8/8 = 1.0$$

Therefore:

$$C_3 = 0.170$$

and:

$$\Delta p = 0.170 \times 4000 = 680 \text{ psf}$$

At the center, from Section C-5:

$$n = m = 4/8 = 0.5$$

Therefore:

$$C_3 = 0.085$$

and:

$$\Delta p = 4 \times 0.085 \times 4000 = 1360 \text{ psf}$$

The average induced pressure, then, is:

$$\frac{680 + 1360}{2} = 1020 \text{ psf}$$

3. The three methods used to compute the mid-depth pressure induced by the design load of the footing are in very close agreement in this particular case. Such close agreement will not always occur. There are so many intangibles and so many assumptions from which each of these methods was derived that there is no way of knowing which method is the more correct.

It has been the experience of the author that design based on method 2b provides for the adequate consideration of settlement under normal conditions of design and construction. Therefore:

$$p_f = 1298 + 1000 = 2298 \text{ psf}$$

Example 4-14 (Continuing Example 4-11)

Required: To compute the anticipated settlement in the layer of soil extending from the footing to the 10% pressure gradient line, in accordance with the provisions of Section 4-5.

It is first necessary to determine the void ratio corresponding to p_f. An enlargement of the e-log p curve is shown in Figure 4-21.

Assuming the e-log p curve to be a straight line, then:

$$\frac{y}{0.560 - 0.539} = \frac{\log 2.0 - \log 1.119}{\log 2.0 - \log 1.0}$$

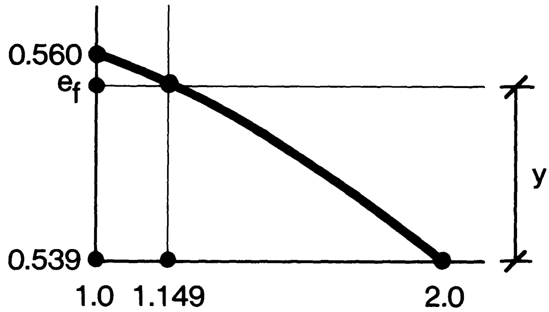


FIGURE 4-21.

From which:

$$y = 0.017 \quad \text{and} \quad e_f = 0.556$$

It should be noted that this value of e_f is on the conservative side because the actual curve is somewhat above a straight line. This will result in computed

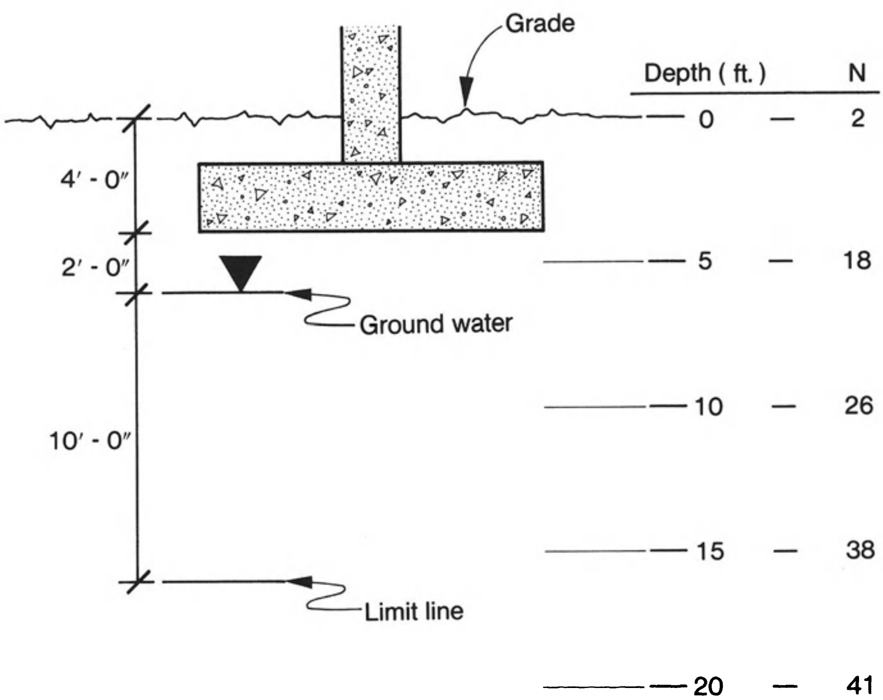


FIGURE 4-22.

settlements slightly higher than those actually realized. The anticipated settlement is, from Formula (4-9):

$$\Delta H = \left[\frac{0.572 - 0.556}{1 + 0.572}\right] 16 \times 12 = 1.95 \text{ inches}$$

Example 4-15

Required: To determine the effect of ground water on the blow counts recorded during a standard penetration test.
Given: A 12 foot square footing situated 4 feet below grade, with ground water at a depth of 12 feet. Blow counts as given in Figure 4-22.

Refer to Figure 4-8. Ground water occurs in level 4, therefore:

$$C_w = \frac{2}{2 \times 12} + 0.5 = 0.58$$

Only those blow counts which 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 4-6.

TABLE 4-6. Blow Count Modification Due to the Effect of Ground Water.

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 4-16

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, all as illustrated in Figure 4-23.

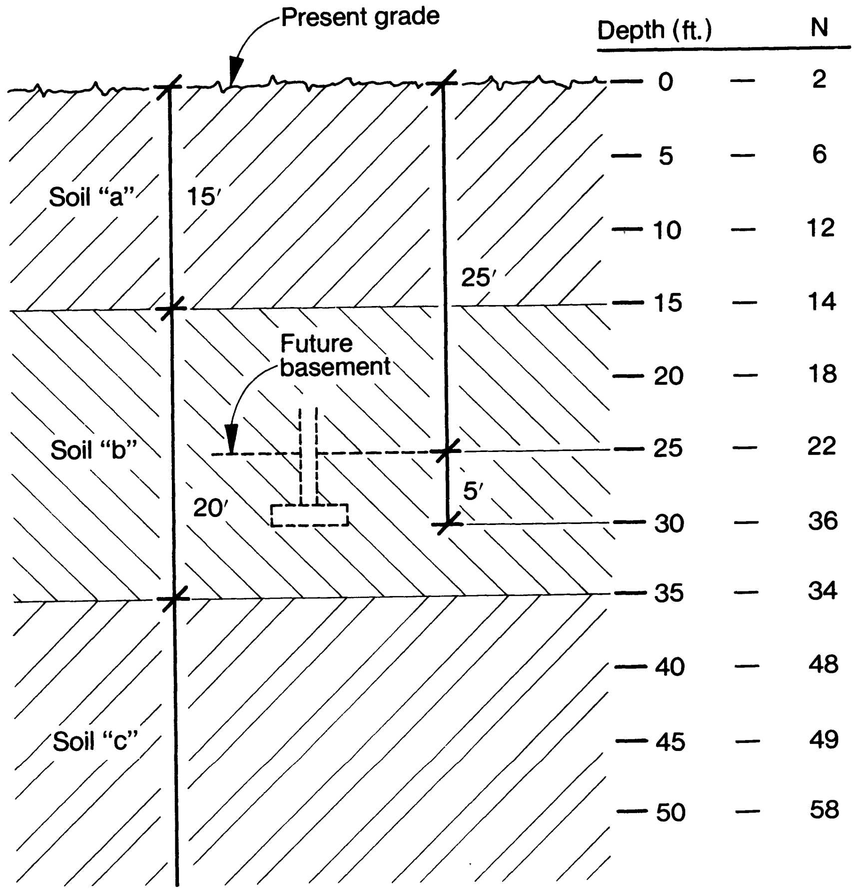


FIGURE 4-23.

Soil properties are as follows:

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

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. It is recommended that 25 feet be used as the effective overburden.

$$\text{The weight of overburden } p_o = 115 \times 15 + 126 \times 10 = 2985 \text{ psf}$$

TABLE 4-7. Blow Count Modification Due to the Effect of Overburden.

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 effect of overburden.

From Figure 4-9, $C_n = 0.87$. The use of this coefficient is shown in Table 4-7.

Example 4-17

Required: To compute the safe load carrying capacity of a 7′-0″ square footing, subject to the conditions shown in Figure 4-24. The soil is a medium, gray sand, having a unit weight of 115 pcf.

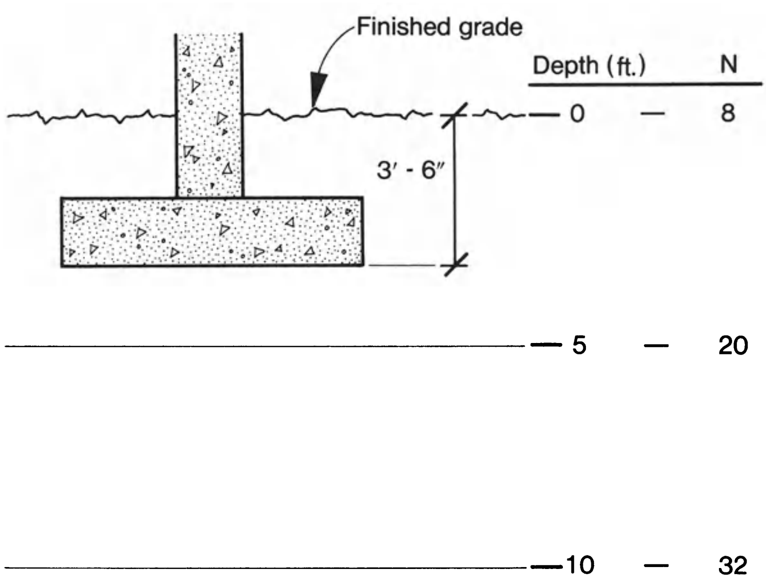


FIGURE 4-24.

The blow count directly beneath the footing is first be determined. It is assumed that a straight line interpolation is satisfactory.

$$N = \frac{3.5}{5}(20 - 8) + 8 = 16.4$$

Referring to Figure 4-12(b), with:

$$D_f/B = 3.5/7.0 = 0.5 \qquad N = 16.4 \quad \text{and} \quad B = 7 \text{ feet}$$

It can be seen that the soil pressure is governed by settlement, therefore:

$$q_a = 0.1 \times 16.4 = 1.64 \text{ tsf} = 3280 \text{ psf}$$

and

$$P = 3280 \times 7.0 \times 7.0 = 160720\ \# = 161 \text{ kips}$$

5

Spread Footings

5-1. GENERAL

A spread footing may be defined as a reinforced concrete footing poured into an excavation which has been dug by hand or by machine but which has been trimmed by hand to the specified dimensions. The primary purpose of this footing is to transfer vertical and horizontal loads from the building into the ground. 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.

5-2. 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 two purposes:

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

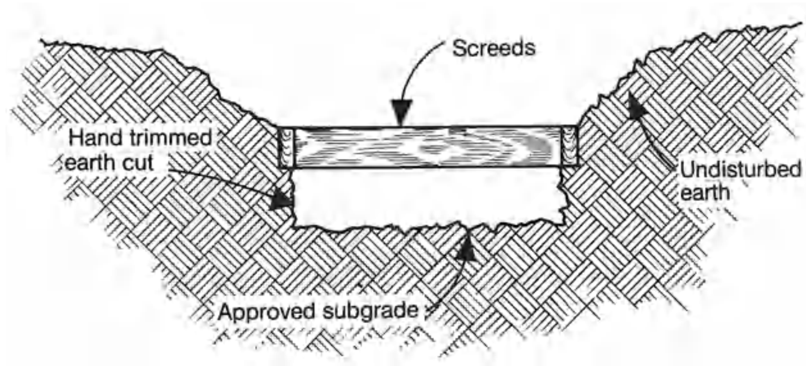


FIGURE 5-1. Footings poured in earth forms.

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 5-1.

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

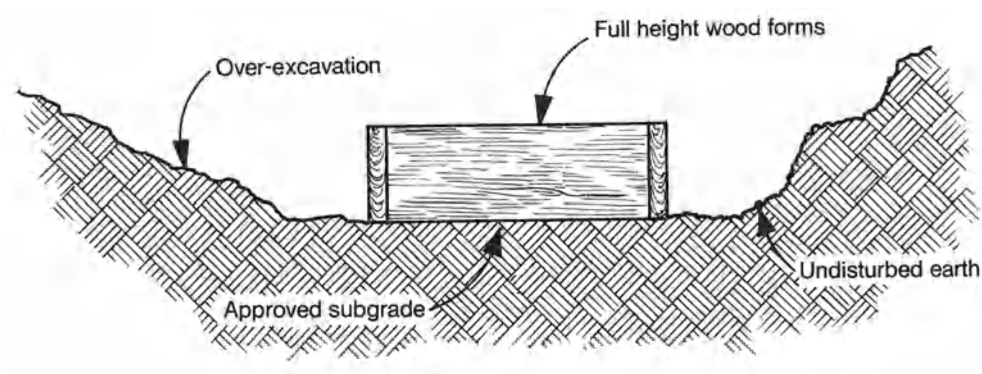


FIGURE 5-2. Footings poured in wood forms.

at the site to insure that adequate forming procedures will be used. In soils whose side walls will fail, the footing area must be overexcavated 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 5-2.

After the footing has been poured and the forms removed, the area of overexcavation must be filled. This can be accomplished in one of two ways:

1. Fill the area with lean concrete
2. Backfill the area with approved soil compacted to an approved density

5-3. APPROVAL OF SUBGRADE

All footings must extend down to and bear on a subgrade inspected by and approved by the architect or engineer of the project. The allowable bearing pressure of 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.

5-4. INTERACTION 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:

$$q_a = \frac{P}{B \times D} \tag{5-1}$$

Where:

q_a = the allowable soil bearing pressure as determined by the principles set forth in Chapter 4

P = the load supported by the footing plus the weight of the footing itself

B and D = the width and length of the footing

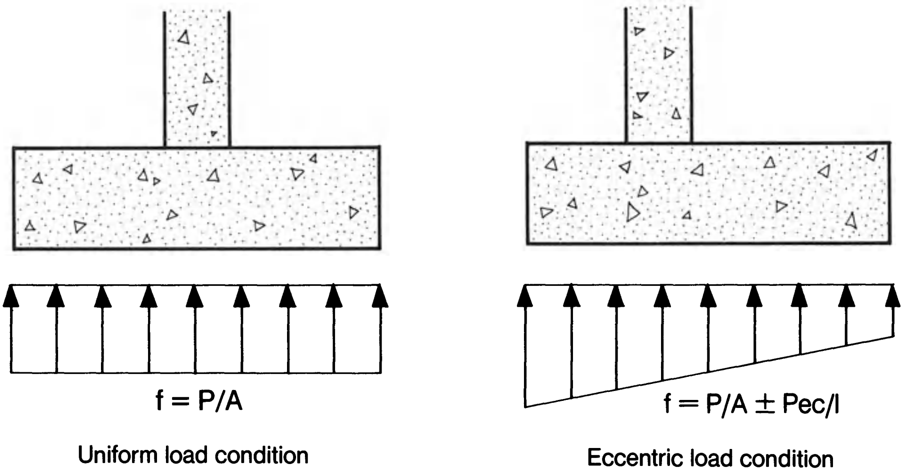


FIGURE 5-3. Difference in soil pressure distribution between footings supporting concentric and eccentric loads.

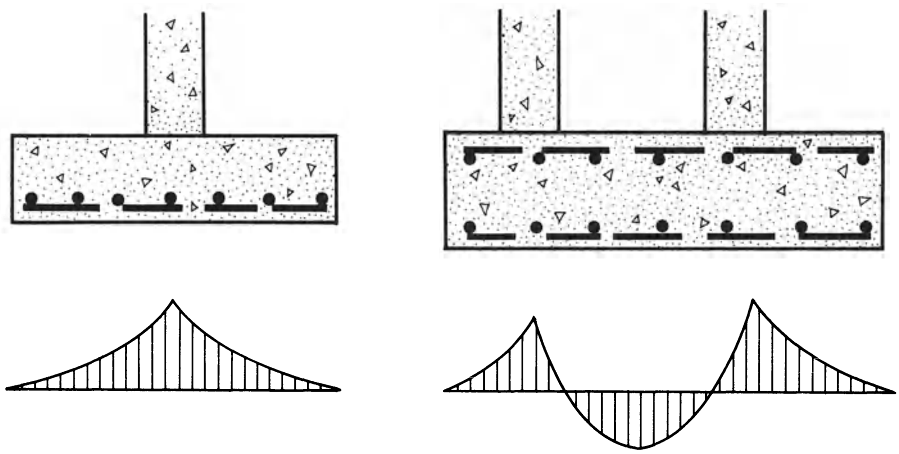
Note: When the footing is continuous, as under a wall, the length of footing is taken as one foot.

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 5-3(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 5-3(b).

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. It should also be forcefully noted that if the eccentricity were to fall outside of the middle third, then the bearing pressure at the other side of the footing would theoretically become tension. Since tension cannot be developed on the surface of contact, a completely different type of design would be required.

5-5. TYPICAL FOOTING REINFORCEMENT

Footings 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 5-4, where the bending moment induced by the soil bearing pressure is plotted on the compression side of the footing.



Single column condition	Multiple column condition

FIGURE 5-4. Typical reinforcing arrangement in footings for single and multiple column conditions.

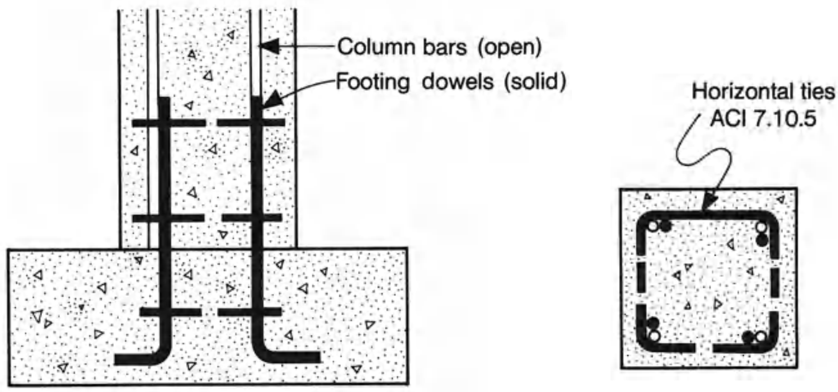
Reinforcing shall be placed in both directions on each of the tension side of the footing, as indicated in Figure 5-4. Bars shall be spaced equally throughout the width of the footing, with the first bar approximately three inches clear from the footing edge. Bars shall be wired together at sufficient 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 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 erosion by the action of soil, water, or air.

5-6. VERTICAL DOWELS

Column Dowels

All columns are reinforced with vertical bars. The force carried by these bars must be transferred into the footing concrete. This can be accomplished in either of two ways:

1. Extend the column bars down into the footing.
2. Use a short piece of bar called a *dowel*. One dowel is required for each column bar. Extend the dowel down into the footing and up into the column. The force in the column bar is then transferred into the dowel, and from the dowel into the footing.



Dowels in section   Dowels in plan

FIGURE 5-5. Dowel placement in footings.

Because it is difficult to hold the relatively long column bars in place before and during the pouring of the footing concrete, the method of dowels is usually preferred by contractors and is usually specified by engineers.

The use of dowels to transfer load from a column to a footing is illustrated in Figure 5-5.

Wall Dowels

Wall dowels are similar to column dowels except that they are usually smaller in diameter and there is more space between adjacent bars. The requirements relative to column dowels are also applicable to wall dowels.

Requirements Relating to Dowels

All requirements relative to dowels, including development lengths, laps, hooks, and bar substitutions, are given in Appendix E.

5-7. TYPICAL FOOTING DETAILS

Purpose

The purpose of a footing is to transfer load from a building or a building element into the ground. The load may be vertical, as in the case of live or dead load, or it may be horizontal, as in the case of wind, earthquake, or earth pressure. Footings can be designed to support walls, individual columns or groups of columns.

Wall Footings

Wall footings are continuous footings which 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.

Footings must be engineered to support and transfer the loads to which they are subjected. 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 5-6.

It should be noted that when the footing must be increased in order to provide more bearing area, or for any other reason, then the size and number of the reinforcing bars must be checked for compliance with the new design requirements.

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. Except for very adverse soil conditions, the stepping arrangement shown in Figure 5-7 will normally prove to be satisfactory.

Particular care must be taken by the contractor to maintain vertical or near vertical cuts when excavating for stepped footings. Refer to item 6 in Section 10-3.

Individual Column Footings

It is important, for the reasons discussed in Section 5-4, that the center of the footing be positioned directly beneath the center of the column. Otherwise the

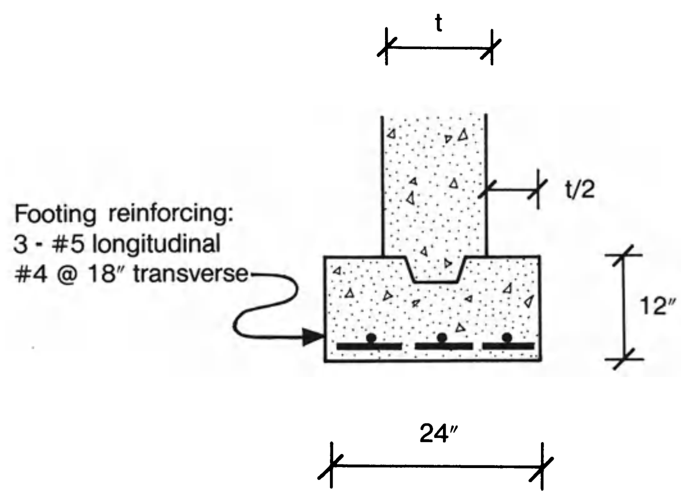


FIGURE 5-6. Recommended minimum size and reinforcing for wall footings.

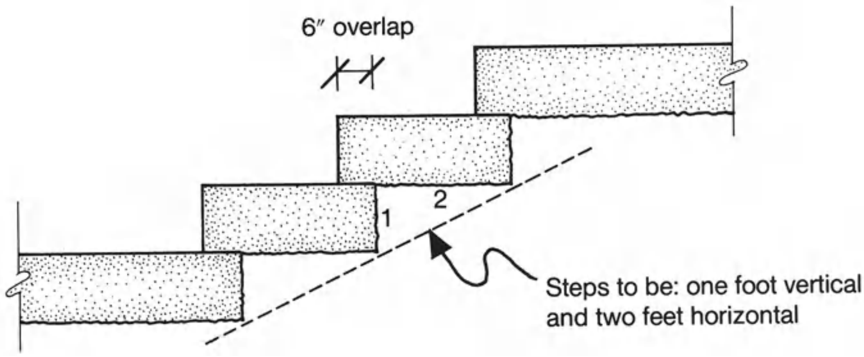


FIGURE 5-7. Recommended detail where wall footings must be stepped.

footing will be eccentrically loaded, and this can lead to very serious consequences. If the footing is overexcavated, however, there is no harm 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 5-8.

These columns cannot be supported on individual footings because they 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

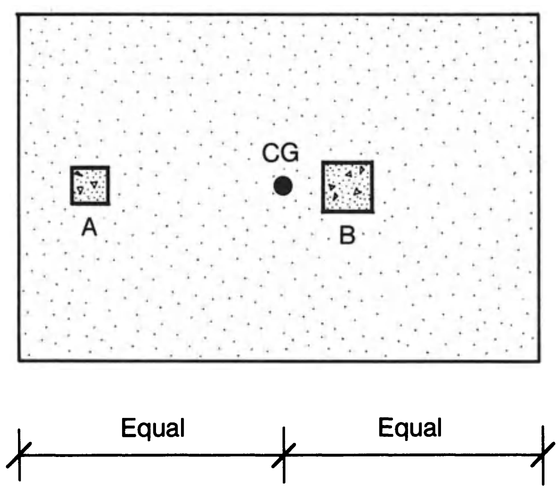


FIGURE 5-8. Single footing supporting multiple columns.

centroid of the footing directly beneath the center of gravity of the loads. The reason for this, of course, is to avoid eccentricity.

Required Footing Area

The required area of contact between a footing and the soil beneath is determined from the following basic formula:

$$\text{Pressure} = \frac{\text{Load}}{\text{Area}}$$

From which:

$$q_a = \frac{Q}{B \times D}$$

Where:

q_a is the allowable soil bearing pressure, psf or ksf
Q is the vertical design load, # or k
A and B are the length and width of the footing, ft

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 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 existance of poor soil at normal bearing elevations, which would not only require the use of large, overlapping footings, but which would also raise the serious question of differential settlement

Mat foundations are usually made a part of the basement floor. They are usually at least 24 inches in thickness, and are heavily reinforced. Design of these mats is essentially that of an inverted flat plate.

5-8. 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 setback from the building line. In such instances there may not be sufficient space in which to position a square footing. There are three alternatives to this problem:

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 5-9. The width and length of the footing shall be computed as follows:

$$B = 2(x - y) \quad \text{and} \quad D = \frac{P}{Bq_a}$$

A rectangular footing supporting a single column should be proportioned so that the length to width ratio does not exceed three. A rectangular footing becomes

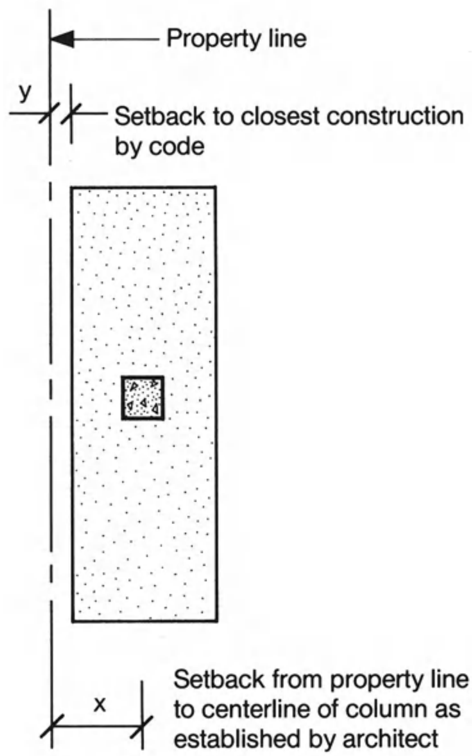


FIGURE 5-9. Rectangular footing solution to problem of column positioned adjacent to property line.

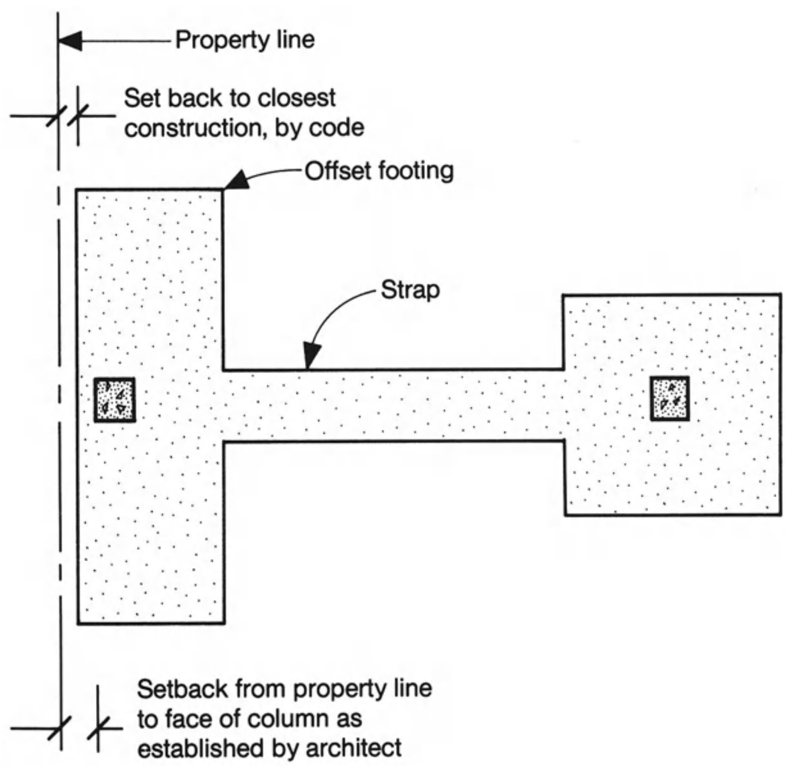


FIGURE 5-10. Strap footing solution to problem of column positioned adjacent to property line.

more of a one-way element as the length to width ratio increases. This type of footing, therefore, will be thicker, more heavily reinforced, and less cost effective.

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 5-10.

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 is shown diagrammatically in Figure 5-3. 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 5-9 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:

1. It can be flush top with the footings, in which case it must be poured monolithically with them.
2. It can be raised up so as to sit directly on the footings, in which case it can be poured subsequent to the footing pour.

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 illustrated in Figure 5-8 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.

5-9. 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 multistory concrete building weighs considerably more than a two or three story light weight steel frame building. It is the nature of soil that its load bearing capacity increases with depth. The footings of the multistory 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 an in-depth discussion of soil bearing pressure, refer to Chapter 4.

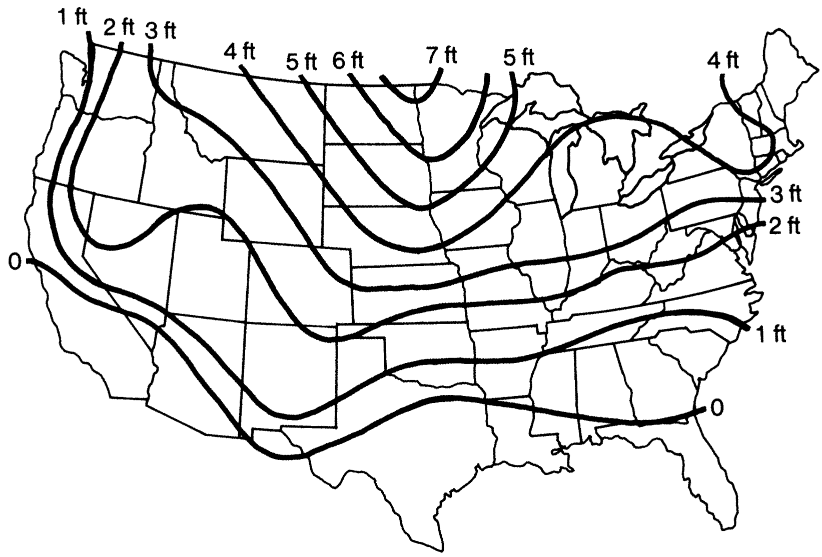


FIGURE 5-11. Maximum anticipated depths of freezing as inferred from city building codes. Actual depths may vary considerably depending on cover, soil, soil moisture, topography, and weather. [Ref. 18]

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 wall is to be built. The information given in Figure 5-11 may be used to gain insight as to the general variation of the frost line depth in different areas of the country. This information, however, should not be used for purposes of 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 Section 11-9, 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 acceptable, cost effective soil beneath.
2. Drive piles or drill piers through the expansive soil, and extend them to the more resistant material normally found at greater depth.

When the additional depth of excavation is no more than 10 to 15 feet, the use of spread footings is usually preferred by contractors as being a faster and more cost effective procedure.

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. The recommended maximum height to length ratio between footings is shown in Figure 5-12.

The proximity between footings is usually measured in terms of slope, as indicated in Figure 5-12. This slope, as defined by the tangent of the angle, is usually limited to 1 : 2. This slope should not be exceeded when excavating in any kind of soil that has predominantly granular characteristics. When excavating in rock or in very dense clay, it may be possible, subject to engineering evaluation, to increase the slope to 1 : 1.

It is noted that the proximity is measured from the top of the lower 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 proximity should be measured from the bottom of the lower footing to the bottom of the upper footing.

These recommendations are only valid provided that the line of excavation does not undercut the proximity line. If this happens then a new proximity line must be established. One segment of this line will extend from the top of the lower

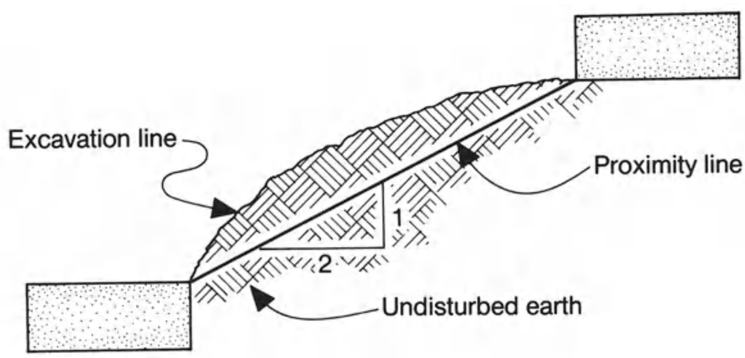


FIGURE 5-12. Recommended proximity line between adjacent footings of different elevation.

footing to the point where the undercut occurs. A second segment must extend from this point to the bottom of the upper footing.

Proximity to Adjacent Properties

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, and which would certainly be costly to defend and to correct, if those in charge of the new project are found at fault.

When there is an existing structure 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 proximity ratio of 1 : 2, as shown in Figure 5-12, may serve as a starting point, but the final ratio must be the result of an engineering evaluation.

Effect of Ground Water

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 extend 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.

The existence of ground water can be determined during the conduct of the subsurface soil exploration, as described in Section 3-5. This exploration, of course, should be made well in advance of design commitment by the architect.

Structures built below the water table are subjected to hydrostatic pressure. In lightly loaded buildings this uplifting pressure can become a formidable obstacle. Refer to Appendix F for further information regarding the phenomenon of buoyancy.

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. Refer to Section 6-16.

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.

5-10. 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 5-5.

5-11. SAMPLE PROBLEMS

In all problems it is assumed that the superimposed load carried by the footing has been increased to account for the weight of the footing. This is one of the several procedures 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 5-1

Required: To determine (1) 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, and (2) the recommended size of footing.

Since there are no apparent restrictions to the proportioning of this footing, it will be made square. The footing design, therefore, will utilize two-way action, which is the natural way for concrete to act. A square footing is also preferable from the standpoint of cost.

From Formula (5-1):

$$6.0 = \frac{250}{A} \quad (\text{Note } q_a = 6.0 \text{ ksf})$$

Therefore $A = 41.7$ square feet and $B = 6.45$ feet

Final design:

$$B = 6'\text{-}6''$$

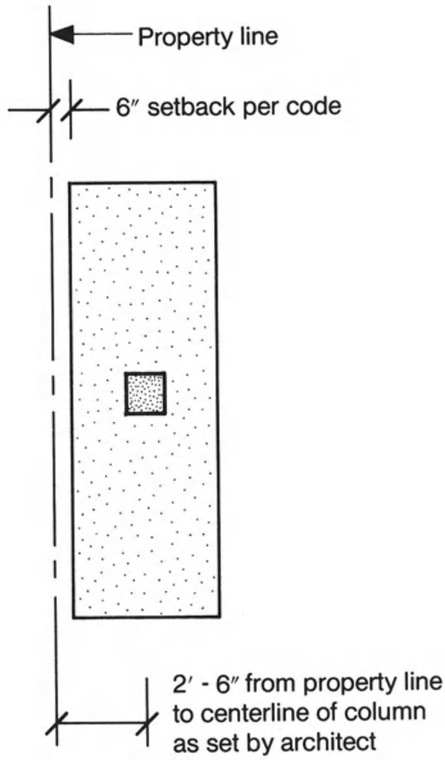


FIGURE 5-14.

Example 5-3

Required: To determine the size of a rectangular footing required to carry a column adjacent to a property line, as given in Figure 5-14. The column load is 250 kips and the allowable soil bearing pressure is 2½ tsf.

The footing width is computed as $B = 2(2.5 - 0.5) = 4.0$ feet. From Formula (5-1):

$$5.0 = \frac{250}{4.0 \times D}$$

Hence $D = 12.5$ feet. The footing, therefore, is 4′-0″ × 12′-6″.

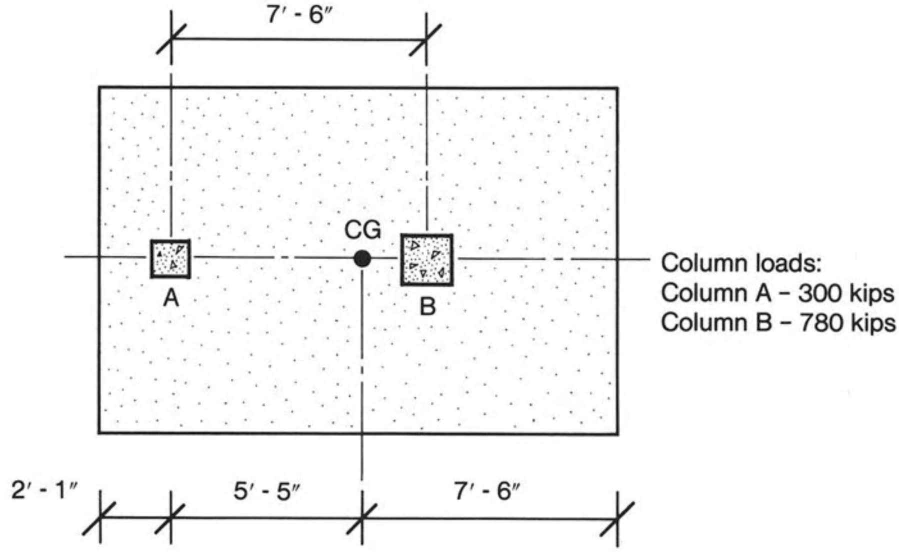


FIGURE 5-13.

Example 5-2

Required: To determine the length and width of a combined footing to carry the columns indicated in Figure 5-13. The allowable soil bearing pressure is 3 tsf.

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, taken from column A, is calculated as follows:

$$x = \frac{780 \times 7.5}{300 + 780} = 5.42 \text{ feet}$$

From Formula (5-1):

$$6.0 = \frac{1080}{A}$$

Therefore $A = 180$ square feet and a 12′-0″ × 15′-0″ footing may be used. Note that there are any number of footing sizes that would satisfy area and clearance requirements.

Example 5-4

Required: To determine the appropriate dimensions for an offset footing assembly, as shown in Figure 5-15. Assume the following column loads: A = 250 kips and B = 400 kips. The allowable soil bearing pressure is 3 tsf. Also develop a simplified shear and bending moment diagram from which the strap could be designed if desired.

From Formula (5-1):

$$6.0 = \frac{267}{B \times D} \quad \text{Use } 4'\text{-}0'' \times 11'\text{-}6'' \text{ at column A}$$

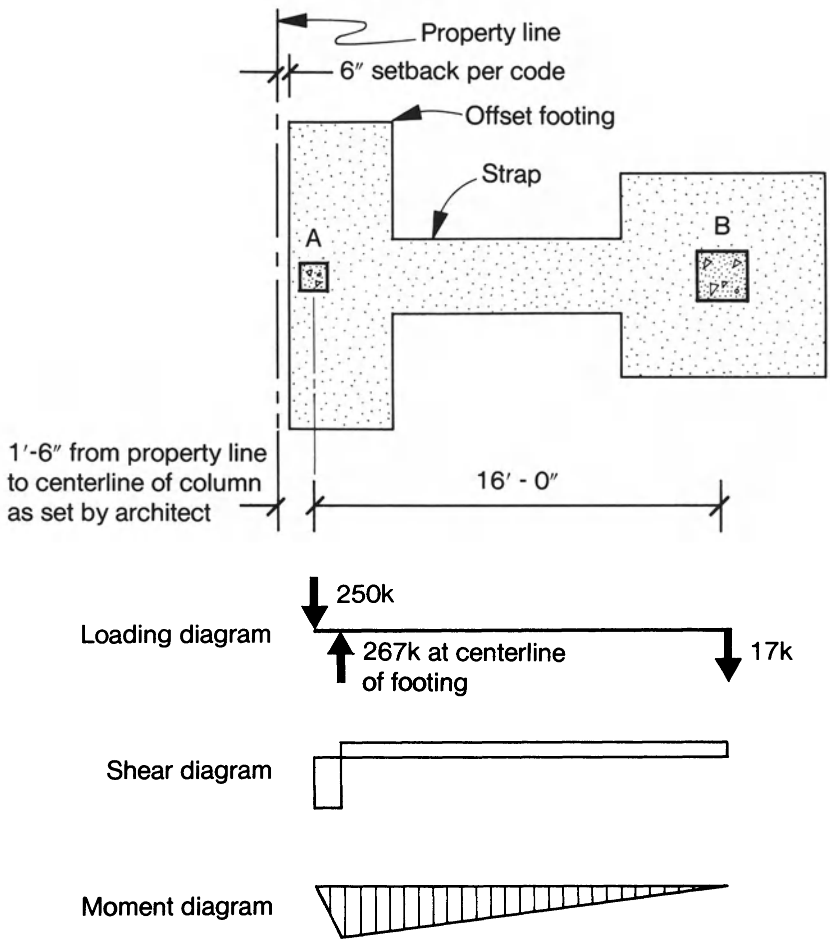


FIGURE 5-15.

and

$$6.0 = \frac{400}{B \times B} \quad \text{Use } 8'\text{-}6'' \times 8'\text{-}6'' \text{ at column B}$$

Example 5-5

Required: To adjust the sizes of the footings listed in Table 5-1, 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.

The procedure is to first determine the size footing required to carry the total load for each column. Secondly, calculate the dead load bearing pressure for each footing. Thirdly, calculate the footing areas which will result in all footings having relatively the same dead load bearing pressure. Note that the footing having the lowest dead load bearing pressure in the group will govern. Finally, select each footing size in accordance with previously determined increments. This procedure is tabulated in Table 5-2.

TABLE 5-1. Design Loads for Footings.

Footing	Dead	Live	Total[a]
1	240	150	390
2	350	140	490
3	600	440	1040
4	410	160	570

[a] All loads are in kips.

TABLE 5-2. Footing Design Based on Dead Load Bearing Pressure.

Footing	(a)	(b)	(c)	(d)
1	48.75	4.92	52.0	7′-3″
2	61.25	5.71	75.8	8′-9″
3	130.00	4.62	130.0	11′-6″
4	71.25	5.75	88.7	9′-6″

(a) is the area required for total load, computed by dividing the total load in kips by 8 ksf.

(b) is the dead load bearing pressure, computed by dividing the dead load in kips by the area computed in (a).

(c) is the new area required after adjustment, computed by dividing the dead load in kips by the least bearing pressure listed in (b).

(d) is the final specified size of the footing, assuming 3″ increments.

6

Piles, Piers, and Caissons

6-1. GENERAL

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 6-1 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 which extend far down into the ground to distribute their load through skin friction to a large mass of weak soil, or to transfer their load by direct bearing to the dense soil or bedrock which must ultimately exist at some depth below ground level. The elements of such a foundation may be piles, or piers, or caissons.

6-2. PILES

The term *pile* implies 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, or a steel shell filled with concrete. 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.

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

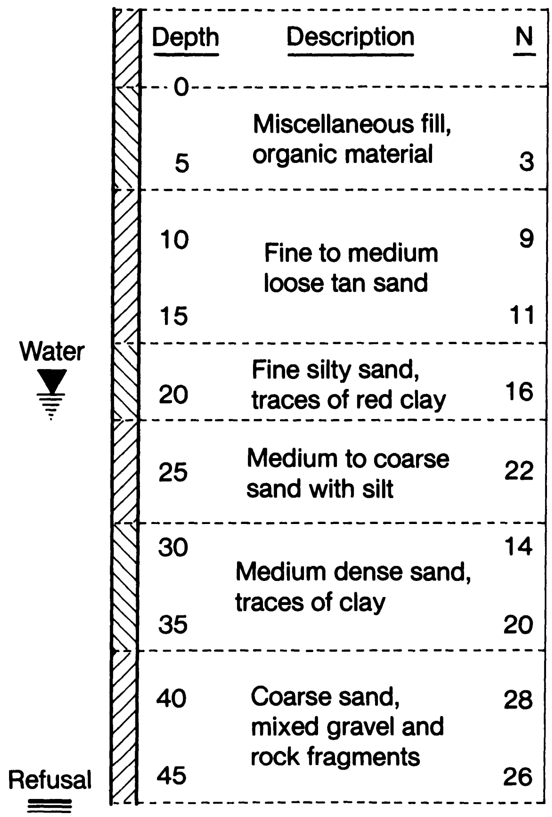


FIGURE 6-1. Sample log of test boring.

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 is a procedure used to facilitate the installation of a pile in sandy soil. In this procedure a 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 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.

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 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.

When timber piles are driven, there is a tendency for the upper part of the pile to split. It is recommended that the upper three feet of each timber pile be strengthened with steel bands to prevent this type of splitting.

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:

$$C = \frac{S - 1.645\ SD}{1.88} = \text{psi} \tag{6-1}$$

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 and SD refer to Table 6-1.
2. Multiply the working stress by the following factor, depending upon the process of conditioning prior to treatment:

Air-dried	1.00
Steam conditioning	0.85

TABLE 6-1. Crushing Strength in Compression Parallel to the Grain for Timber Piles.[a] [Ref. 2]

Species	*S*, psi	*SD*, psi
Douglas fir—coast	3784	734
California red fir	2758	459
Western hemlock	3364	615
Longleaf southern pine	4321	707

[a] For calculations using S and SD, refer to Formula (6-1).

3. For oak piles only, increase the working stress by 10 percent.
4. For douglas fir and southern pine piles only, increase the working stress by $0.2L$ percent, where L is the distance in feet from the tip of the pile to the critical section.

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.

Structural Steel HP Piles

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 which contain hard lenses, boulders, or other obstructions. Due to the ease with which steel can be spliced by welding, a structural steel pile can be extended in length almost without limit.

The shapes used in steel pile construction are identified by the symbol HP, and they 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.*

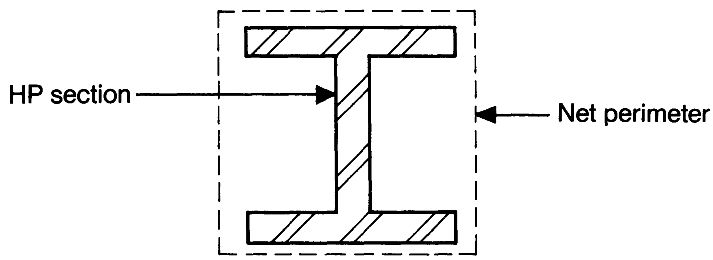


FIGURE 6-2. Net perimeter of HP steel piles. [Ref. 24]

HP piles are manufactured in steel having a minimum yield point of 36 ksi, and 50 ksi yield point is available for special conditions. The allowable working stress on these piles is usually limited by code to $0.35F_y$.

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, and that 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 HP piles are installed as friction piles, the friction is assumed to be developed on the net perimeter of the cross section. This perimeter is indicated in Figure 6-2.

Perimeter lengths of HP piles are given in Table 6-2.

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 Section 8-4.

Steel Shell Piles

Steel shell piles filled with concrete are a very popular mid-length, mid-capacity pile. Pile shells are readily available in 12, 14, 16, and 18 inch butt diameters. The surface of the shell is fluted and is also tapered to provide increased frictional

TABLE 6-2. **Net Perimeter of HP Piles.**

Section	Net Perimeter
HP 14 × 14	4.75 feet
HP 12 × 12	4.00 feet
HP 10 × 10	3.33 feet
HP 8 × 8	2.67 feet

resistance. A forged steel conical nose is factory-attached to the tip of each pile to facilitate driving. End bearing, therefore, is diminished. Because of the reduction in end bearing, these piles must develop almost all of their design strength through friction. 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. After the hollow shell has been driven to proper bearing, the mandrel is withdrawn and the 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.

The maximum load that can be sustained at the butt diameter by this type of pile is given by the following formula:

$$P_{\text{design}} = 0.35F_yA_s + 0.33f'_cA_c \tag{6-2}$$

FIGURE 6-3. Installation of steel shell piles. [Ref. 15]

FIGURE 6-4. The aesthetic use of piles. [Ref. 15]

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.

This formula 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. Therefore:

$$\Delta L = \frac{PL}{AE}\text{ steel} = \frac{PL}{AE}\text{ concrete} \tag{6-3}$$

A typical steel shell pile installation is shown in Figure 6-3.

In bridge construction piles frequently project above the surface of the ground or water below. These piles then become an important consideration in the architectural treatment of the entire area. An example of this use is shown in Figure 6-4.

Steel Pipe

Steel pipe combines certain characteristics of HP piles and steel shell piles. Its driving characteristics are much like those of the HP piles in that it can be driven through difficult stratas and can readily be extended to end bearing. It also shares with the steel shell pile the potential of being filled with concrete. In this instance the maximum load to which the pile can be subjected may be computed from Formula 6-2. Steel pipe is usually manufactured by the seamless or cold formed process, with yield points of 35 and 46 ksi. Properties of all available pipe sizes are given in Part 1 of the *AISC Manual of Steel Construction.*

6-3. PIERS

The term *pier* describes an element in which a continuous flight auger (much like a large corkscrew) is drilled into the ground, after which the earth spoils are removed and the resulting shaft filled with concrete. Because piers are drilled rather than driven, there is considerably less noise and vibration than in a pile installation. It is the nature of all piers that their base is flat ended. They are, therefore, effective in resisting load through end bearing as well as in friction. Modern drilling machinery permits a limited belling out at the base of the pier when in a cohesive soil. Such bells can not be inspected hands-on, but can be inspected only from ground level.

There are three general methods by which a pier can be installed, the selection of which is primarily a function of the type of soil through which the pier must be extended.

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 method used. 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. 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. The hole is then filled with concrete with the use of a tremie or an elephant trunk. Free fall must not be permitted. 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 significantly densify the soil. 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 which are installed by driving.

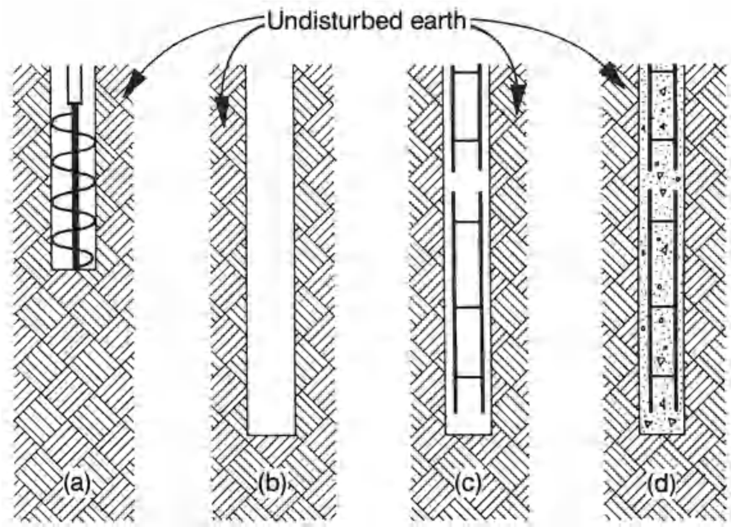


FIGURE 6-5. Installation of a pier in clay, using continuous flight auger.

Pier installation by this method is only applicable when the open hole will remain essentially dry. A high water table or excessive seepage of water can render this method impractical.

A typical pier installation using this method is illustrated in Figure 6-5: (a) The auger is advanced and withdrawn in stages until completion of the hole. (b) The hole has been extended to contract depth and is cleaned of any debris. (c) Reinforcing can be placed at this time, if required. (d) The hole is concreted, using a tremie or elephant trunk.

Installation in Sand, with Slurry

This method of pier installation is similar to that described for clay except that a slurry of bentonite and water 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 Section 11-3, 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 considerable lateral pressure against the side walls of the hole. The slurry is continuously introduced into the open hole during the 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 disposed. Care must be taken in the disposal of

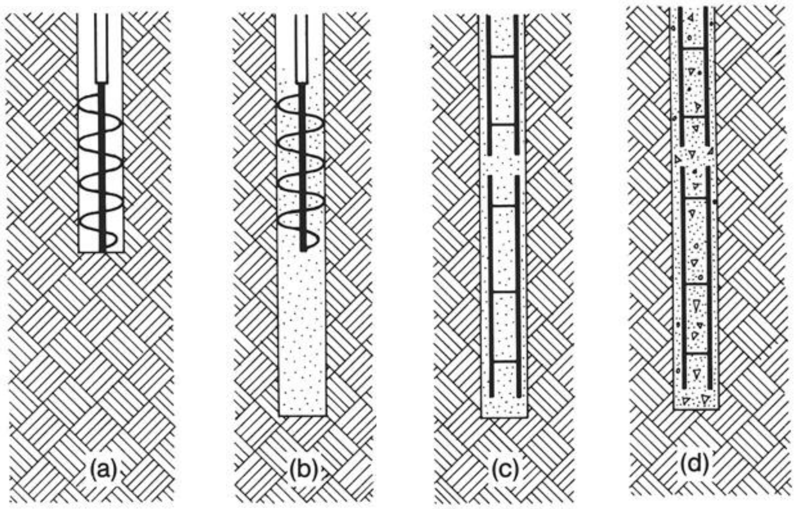


FIGURE 6-6. Installation of a pier in sand, using continuous flight auger and a slurry of bentonite.

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 pier installation is illustrated in Figure 6-6: (a) The auger advances hole to contract depth while continuously adding slurry. (b) The auger, along with all earth spoils, is withdrawn. (c) Reinforcing can be placed at this time, if required. (d) The hole is filled with concrete, which displaces the slurry to the surface.

Installation in Sand, with Hollow Shaft Auger

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.

A frequently used method by which a pier can be installed in sandy soil is to use a special type of auger, the shaft of which is hollow. 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

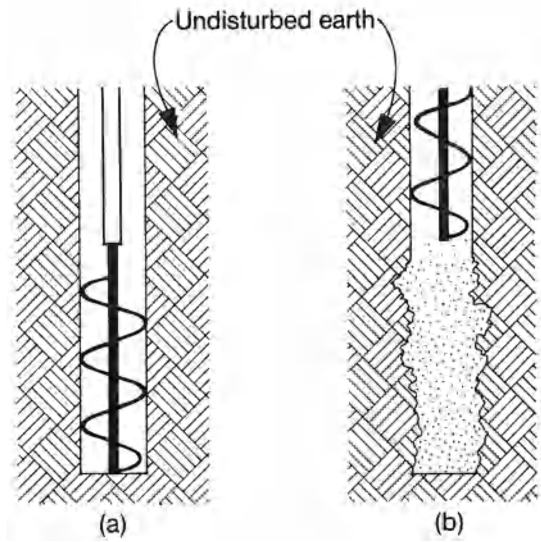


FIGURE 6-7. Installation of a pier in sand, using a hollow shaft auger.

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 6-7: (a) The auger is advanced to contract depth. No earth spoils are removed. (b) Grout is pumped through the hollow shaft in the auger, forcing the auger and earth spoils to the surface. Note that the grout pushes against the side walls, filling any voids and fissures and densifying any soft spots.

6-4. CAISSONS

The term *caisson* is reserved for elements of large diameter, usually 24 to 48 inches, which are drilled into the ground much like a pier, but are then belled out to provide additional bearing area at the base. A typical caisson is illustrated in Figure 6-8.

The augers used for drilling the shaft of a caisson are substantial pieces of equipment, as illustrated in Figure 6-9. These particular augers are used for drilling in earth. Augers can also be equipped with carbide teeth for use in drilling in rock.

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 whose side walls will not stand or in soils whose 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

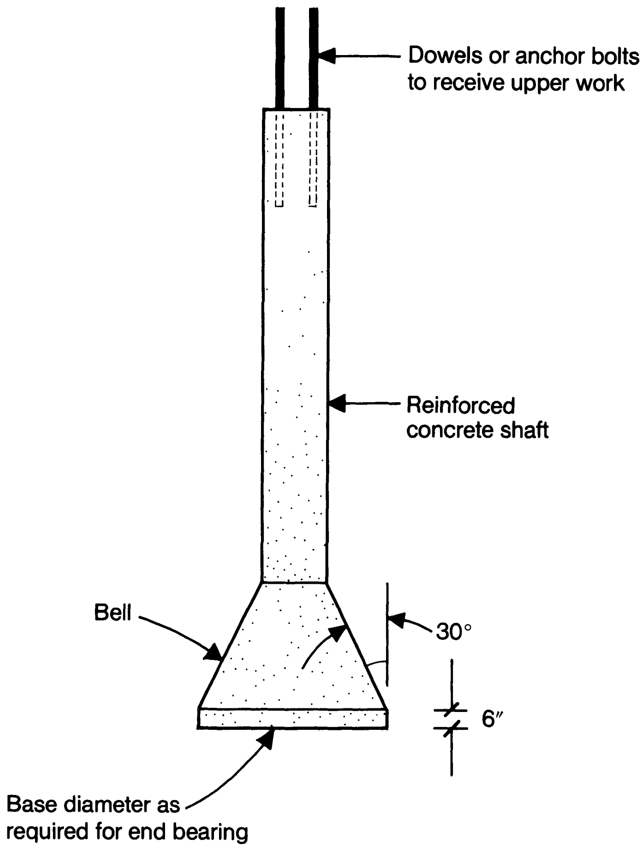


FIGURE 6-8. Typical caisson detail.

can be started by machine but the work must ultimately be completed by hand. It is, therefore, a safety requirement that temporary steel liners be installed in the shaft as the work progresses. This safety precaution must be taken, regardless of how stable the side walls of the hole appear to be.

The type of liners used in this work are illustrated in Figure 6-10.

When there is ground water, these liners can be welded so as to provide an essentially watertight 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.

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.

Prior to 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

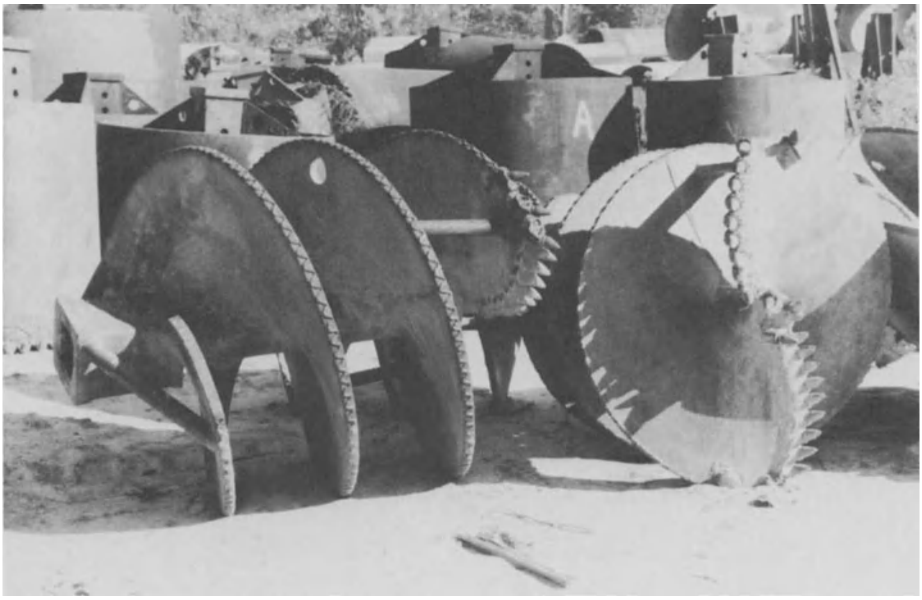


FIGURE 6-9. A large diameter earth auger, used in pier and caisson construction. [Ref. 14]

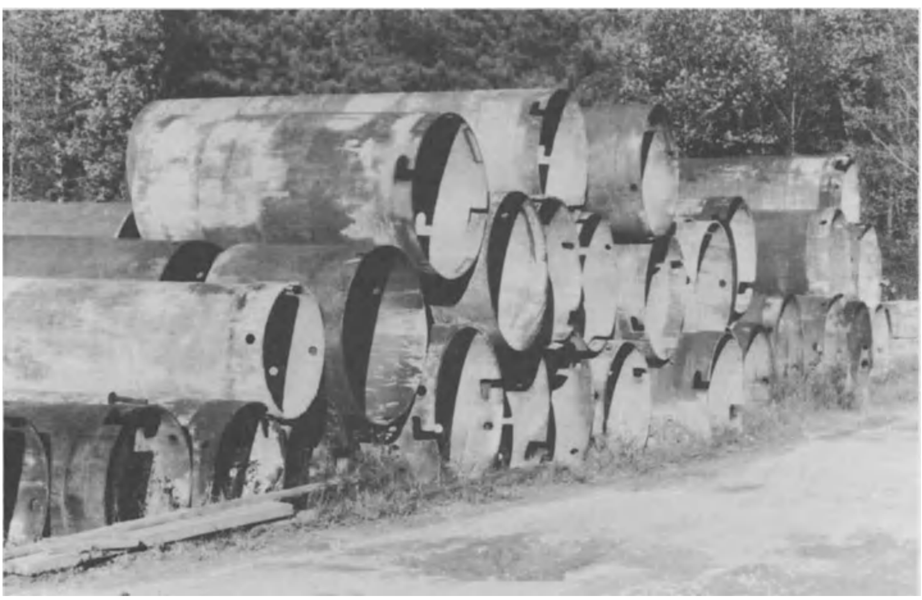


FIGURE 6-10. Steel liners, used in caisson installation to prevent side wall collapse and to protect workmen. [Ref. 14]

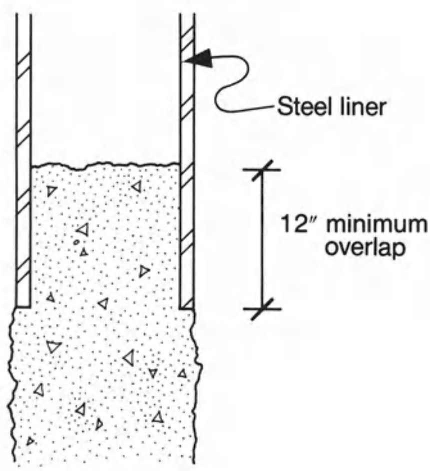


FIGURE 6-11. Removal procedure of steel liner in caisson work to prevent spillage of earth into an unguarded excavation.

should always lag behind the depositing of concrete to insure the stability of the side walls. This is illustrated in Figure 6-11.

The primary purpose of a caisson is to transfer heavy building loads to very dense soil or bedrock by direct bearing. Skin friction is also developed between the caisson shaft and the surrounding earth. It has been the experience of the writer, however, that the effect of friction is generally ignored in the interest of simplifying the calculations.

6-5. 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 which follow draw on several of those different ways, and on the experience of the author.

There is general agreement that these elements derive their load carrying ability from two sources:

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 *adhesion* when referring to cohesive soils.
2. End bearing between the tip of the element and the soil upon which it bears.

The ultimate capacity may be computed numerically by the following formula:

$$Q_{\text{ultimate}} = Q_{\text{shear}} + Q_{\text{bearing}} \tag{6-4}$$

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 (6-4) is illustrated graphically in Figure 6-12.

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 requires that there be a slight slippage between the foundation and the adjacent soil. When the foundation has been extended to refusal it is very doubtful that sufficient slippage will occur. In this case the foundation should be designed solely on the basis of end bearing.

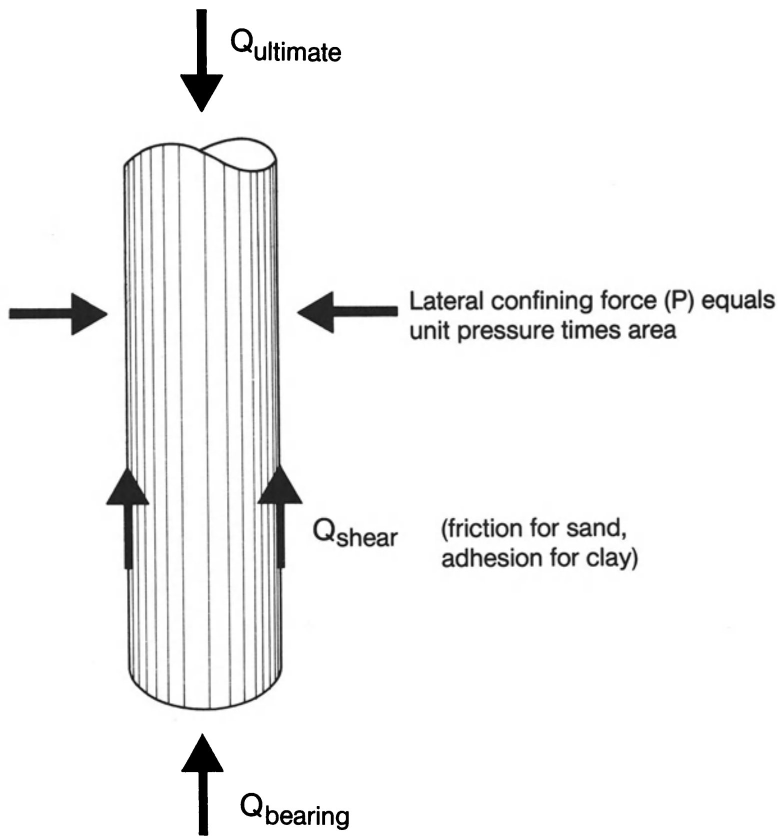


FIGURE 6-12. Factors contributing to the ultimate load resistance of a pile, pier, or caisson.

6-6. UNIT SHEAR STRESS DUE TO COHESION—CLAY

The unit shear stress developed by cohesion along the surface of contact between the shaft of a pile, pier, or caisson and the surrounding clay may be computed from the following formula:

$$f = \lambda c \tag{6-5}$$

Where:

f the unit shear, psf,
λ is a cohesion reduction factor, and
c is the unit cohesion, usually taken as being equal to one-half of the unconfined compression strength q_u.

The numerical value of the cohesion reduction factor λ is primarily dependent upon the following things:

1. The method by which the member is installed. Remember, piles are driven, piers and caissons are drilled.
2. If the member was drilled, was the drilling done in a hole that was dry, or was a bentonite slurry added to control cave-in or influx of water?
3. The consistency of the clay.

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. The cohesion reduction factors which follow reflect those which can normally be anticipated after recovery [Ref. 18]:

For HP piles:	1.0 for all soils
For other piles:	0.5 to 1.0 for soft, sensitive clays
	0.2 to 0.4 for stiff or cemented clays
For piers and caissons:	0.5 with a dry hole, but f shall not exceed 1800 psf
	0.3 with a bentonite slurry,

The reduction factors for piles driven in clay are, to a certain extent, a function of the consistency of the soil. This is also true of blow count. A correlation between the reduction factor and blow count, as recommended by the author, is given in Table 6-3.

In drilled piers and caissons 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 five feet is recommended.

TABLE 6-3. Correlation between Blow Count and Cohesion Reduction Factor for Piles Driven into Clay.

Blow Count N	λ
3	1.0
6	0.8
9	0.6
12	0.4
15	0.2

6-7. UNIT 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, pier, or caisson and the surrounding sand, is illustrated graphically in Figure 6-13.

The unit shearing stress thus developed can be computed from the following formula:

$$f = \gamma D_f K \tan \delta = \text{psf} \tag{6-6}$$

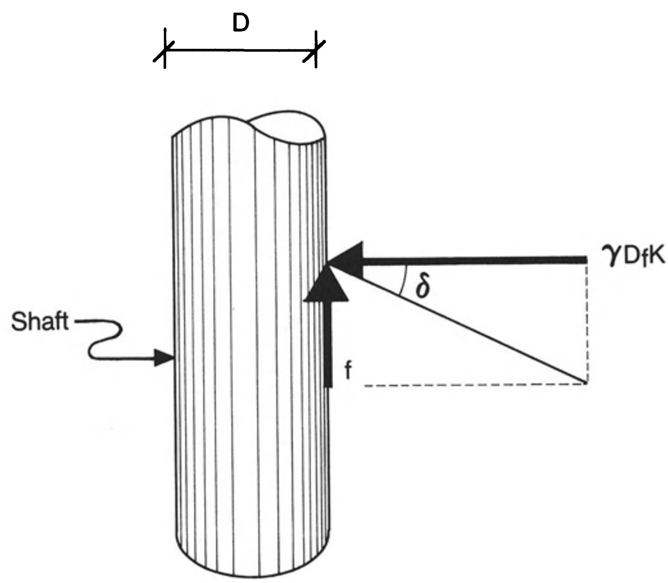


FIGURE 6-13. The determination of skin friction.

TABLE 6-4. Coefficient of Friction between Foundation and Sand. [Ref. 13]

Material	$\tan\delta$
Concrete	0.45
Wood	0.4
Smooth steel	0.2
Rough, rusted steel	0.4
Corrugated steel	use $\tan\phi$

Where:

γ is the unit weight of the in-situ soil,
D_f is the distance below the surface of the earth to the depth at which the unit shear is to be calculated,
γD_f is the vertical unit pressure, usually limited in value to the product of the soil density times 15 shaft diameters,
K is the lateral pressure coefficient, as identified herein, and
$\tan\delta$ is the coefficient of friction between the soil and the shaft, as evaluated in Table 6-4.

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 or caissons (drilled). This is due to the densification caused by the driving operation.

Conservative values for this coefficient are as follows [Ref. 18]:

For piles: Use K = 0.3 for silt
0.5 for loose sand
1.0 for dense sand
2.0 for pile clusters

For piers and caissons: Use $K = K_o = 1 - \sin\phi$ where K_o is the coefficient of lateral pressure for soil at rest

6-8. UNIT END BEARING STRESS—CLAY

The ultimate unit stress developed by end bearing at the bottom of a pile, pier, or caisson end bearing on clay may be computed from the following formula:

$$q_d = cN_c, \text{ psf} \tag{6-7}$$

TABLE 6-5. Bearing Capacity Factor N_c for Piles, Piers, and Caissons.

N[a]	Consistency	Cohesion, psf	N_c
2–4	Soft	250–500	6
4–8	Medium	500–1000	7
8–15	Stiff	1000–2000	8
15–30	Very stiff	2000–4000	9
>30	Hard	>4000	10

[a] Blow count, taken from boring log.

Where:

q_d is the ultimate unit stress,
c is the minimum value of cohesion within a height of several feet above and below the foundation base, and
N_c is a bearing capacity factor, originally introduced in Section 4-3.

The value of N_c, as now related to deep foundations, varies from about 6 to 10, depending on the stiffness of the clay. A value of 9 is conventionally used [Ref. 13].

It was shown in Tables 2-4 and 3-2 that there is a general relationship between blow count, as recorded during the test borings, soil consistency, and soil cohesion. The bearing capacity factor N_c is similarly related. Values for this factor, as recommended by the author, are given in Table 6-5.

6-9. UNIT END BEARING STRESS—SAND

The ultimate unit stress developed by end bearing at the bottom of a pile, pier, or caisson end bearing on sand, may be computed from the following formula:

$$q_d = 0.4\gamma B N_\gamma + \gamma D_f N_q, \text{ psf} \tag{6-8}$$

Where:

q_d is the unit end bearing compression stress is the unit weight of the soil,
B is the side or diameter of the foundation,
γD_f is the vertical unit pressure, whose value is limited to that given in Section 6-7, and
N_γ and N_q are bearing capacity factors, originally introduced in Section 4-3.

It is generally recognized that the numerical value of the bearing capacity factors should be somewhat larger for deep foundations than for spread footings. Due to the wide variation in proposals, however, it is recommended that the shallow footing values from Figure 4-2 be used.

In almost all cases the first term in Formula (6-8) is very small compared to the second term. Because of this inequality, the ultimate unit stress in end bearing is usually computed, conservatively, as follows:

$$q_d = \gamma D_f N_q \tag{6-9}$$

6-10. EVALUATION OF DESIGN BY FORMULA

Piles and piers whose load carrying capacity depends on (a) frictional resistance or (b) end bearing on soil other than refusal, should not be designed solely on formulas derived from theory. The reasons for this are as follows:

1. The technology from which the formulas were developed is not standardized. There is a diversity of opinions, even among the experts.
2. The formulas assume that the soil within a given layer is homogeneous, so that average values of the various properties can 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. Deep foundations are assumed to be laterally braced by the adjacent soil. 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. The stronger the soil, however, the truer will be this assumption. Load tests resolve this problem completely.
5. Deflection may be a critical factor in the design. Foundations based on end bearing offer more assurance to the adequacy of deflection computations than do those dependent upon frictional resistance.
6. The judgment of the engineer who designs a pile or pier foundation is penalized because he cannot examine the supporting soil in its natural state. With caissons, on the other hand, the engineer can readily and safely be lowered to the bottom of the shaft where he can directly examine the soil upon which the caisson will bear.

The adequacy of the theoretical design for the previously noted foundations should be substantiated by on-site load tests, as described in Section 6-11.

6-11. LOAD TESTS

Load Test Scheduling

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 were installed. This is the preferred procedure, and it will ultimately result in an overall cost saving to the owner. 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 project. The project specifications must then include a procedure whereby any indicated changes to the layout or design of the foundations 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 in order to give the soil time to reconsolidate and regain its strength. When piles are driven into sand the soil is densified and there is a temporary indication of an 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:

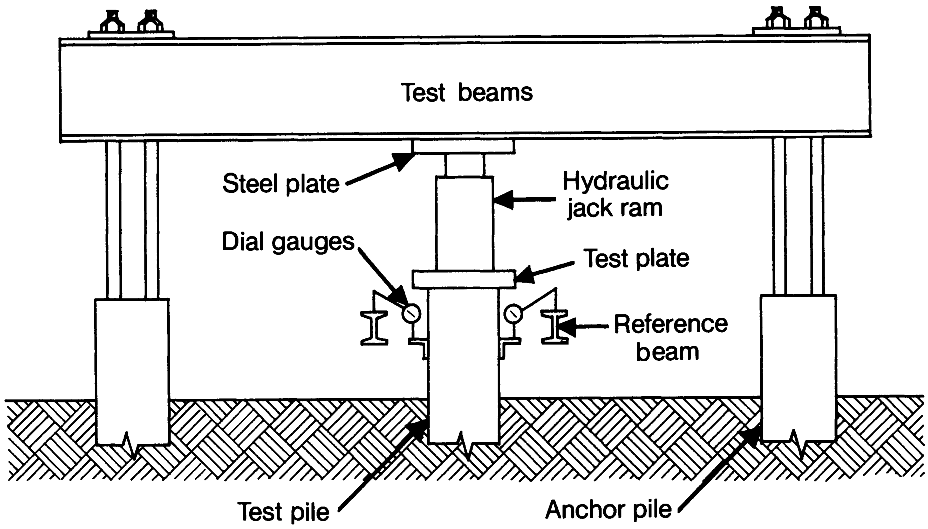
ASTM Test Method D-1143: Method of Testing Piles Under Static Axial Compressive Load

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.

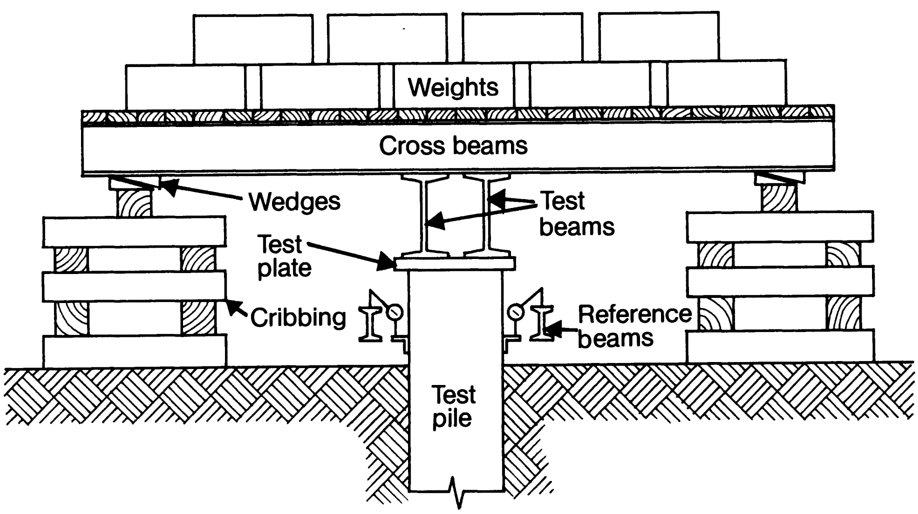
In what is called the Standard Loading Procedure, the pile is loaded to 200 percent of the design load, using 25 percent increments. Provided that the pile has not yet failed, the load is then removed, using 25 percent decrements. The vertical settlement or rebound of the pile is accurately monitored 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 different methods by which the test piles can be loaded, two of which are illustrated in Figure 6-14.

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 be constructed to protect the test site from direct sunlight.



(a) Using hydraulic jack acting against anchored reaction frame



(b) Using weighted platform

FIGURE 6-14. Schematic set-up for a load test on a pile. [Ref. 2]

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, 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 either of the following criteria:

1. The load which produces a predetermined settlement, usually one-half to one inch.
2. The load at which there is a disproportionate increase in the load-settlement curve. This is 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 is illustrated in Figure 6-15, in which ultimate strength is determined as in item 1.

Another load-settlement curve is illustrated in Figure 6-16, in which ultimate strength is determined as in item 2. Note that when plotted to logarithm scale, the curve more closely approximates a straight line.

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 in pile design usually vary between 2 and 3, depending upon circumstances. Factors which must be considered in this determination are

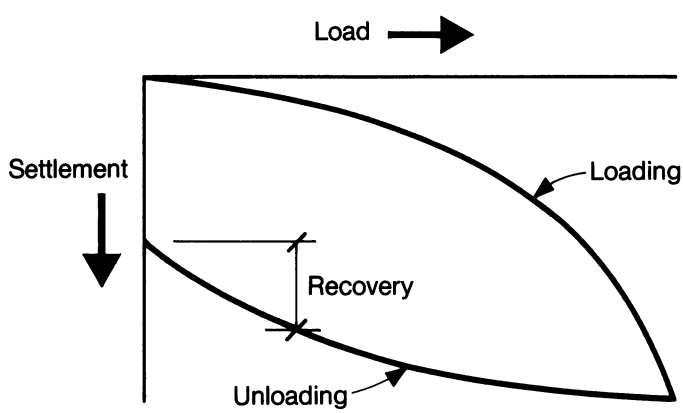


FIGURE 6-15. Load-settlement curve for a pile—arithmetic scale.

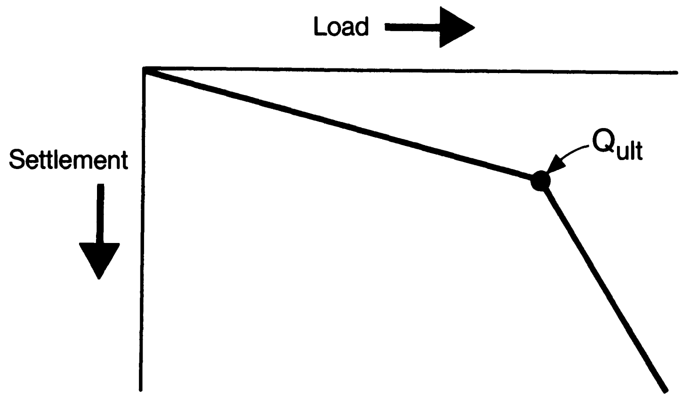


FIGURE 6-16. Load-settlement curve for a pile—logarithmic scale.

the uniformity of the soil, and the properties of the soil throughout the site; the correlation of values between load tests, test borings, and laboratory analysis; and the incidence of ground water.

Safety factors should be higher when there are more intangibles in the design. Surely, a pile bearing on bedrock is a more positive thing than a pile which develops its resistance solely through friction. Based on that premise, safety factors should be increased in accordance with the following sequence:

1. End bearing on refusal
2. End bearing on strong soil or rock, but not refusal
3. Friction with, or without end bearing on relatively weak soil

6-12. 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*. Due to 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 6-17. The center to center spacing between individual piles is normally established as the greater of three diameters or three feet. This dimension is fairly typical for piles and 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.

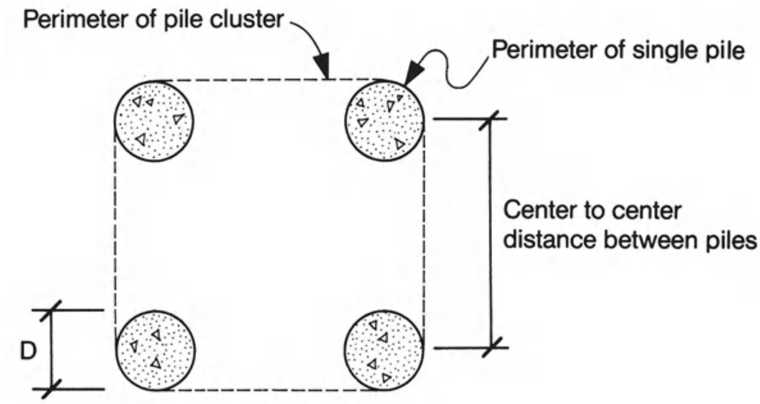


FIGURE 6-17. Typical arrangement of piles in a cluster.

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
$D > 1$	$4\pi D$	$<$	$\pi D + 12D$
$D < 1$	$4\pi D$	$<$	$\pi D + 12$

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.

This comparison was for a four pile cluster. Similar comparisons can be made for other clusters. The result of such an examination can be expressed in the following rule: *The capacity of a pile cluster shall be calculated as the combined capacity of the individual piles.*

Minimum Number of Piles

Piles may wander off dead center while being driven. The resultant eccentricity of load may 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.
3. Provide a three pile cluster, which is self bracing.

A general detail of item 2 is shown in Figure 6-18.

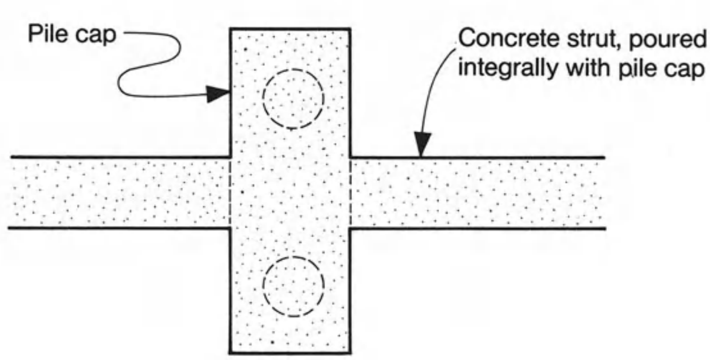


FIGURE 6-18. Typical method of pile cap bracing.

Note that the problems caused by unintentional eccentricity 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 easier to control procedure. Even so, it may still 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.

6-13. BATTERED PILES

Piles may be battered for either of the following reasons:

1. To support lateral loads in addition to vertical loads
2. To provide lateral stability

Typical details of battered piles are shown in Figure 6-19: (a) an arrangement which may be used to support lateral loads acting in the direction toward the battered pile, thereby placing that pile in compression; (b) an arrangement which may be used to support lateral loads acting in either direction; this is also an excellent arrangement where substantial lateral bracing is required.

Piers may also be battered, with details essentially the same as those for piles. Caissons, on the other hand, cannot feasibly be battered.

6-14. 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, so that they can be used as a positive means of providing lateral bracing for the structure above.
7. Piles driven into highly sensitive clays may liquefy the soil, as described in Section 2-13. Liquefaction may also occur in saturated, loose sands. Liquefaction 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 liquefaction.

Piers

8. 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.
9. Piers usually have a larger diameter than piles. For a given length, therefore, piers will develop more frictional load carrying capacity.

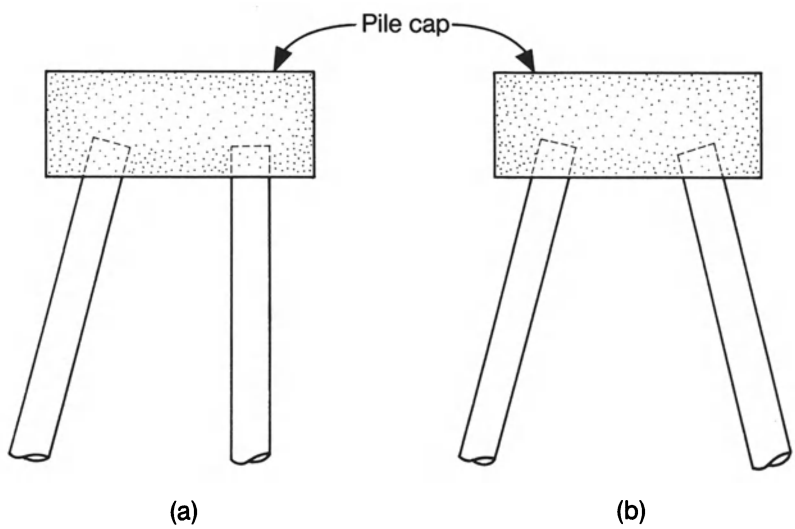


FIGURE 6-19. Battered pile arrangement.

10. Pier construction results in a certain volume of earth spoils, which must be properly used or disposed of.
11. Bad weather can adversely affect the installation of piers more than of piles. This is particularly due to the handling of earth spoils and concrete.

Caissons

12. Caissons are essentially large piers, and generally exhibit all of the advantages and disadvantages of piers.
13. Because caissons are large, they can be readily inspected. This eliminates the guesswork which is always present to a degree in the use of piles or piers.
14. Caissons can be belled out to provide additional bearing area. For this reason they can carry extremely heavy loads.

6-15. 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 with depth
3. When the footing is not required to resist uplift

For other conditions, as in the list which follows, the use of piles, piers or caissons may be appropriate.

4. When soil is very loose, as in the case of sand; or soft, as in the case of clay.
5. When the strength of the underlying soil does not increase sufficiently with depth.
6. When the strata upon which the spread footings would normally bear consists of expansive clay.
7. When soil below the normal bearing strata is subject to variation or seasonal change in water content.
8. When very hard material is relatively close to the surface of the ground, although lower than what would be customary for spread footings. In this instance piers or caissons should be seriously considered.
9. When the site is covered by a thick layer of miscellaneous fill.
10. 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 subbasement 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 then foundations must consist of piers or caissons drilled down to bedrock.

11. Piles can be very successfully used to densify the soil. The vibration caused by the driving operation acts as a densifier. Since the pile also displaces a certain volume of soil this further densifies the soil by causing a reduction in void ratio.

6-16. CONCRETING WITH TREMIE OR ELEPHANT TRUNK

Concrete is normally deposited into an open caisson excavation in the following sequence:

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 is considered to be too high. This occurs occasionally with spread footings and is a normal occurrence in the concreting of piers and caissons.
2. When the concrete must be deposited through a standing head of water.

For each of these two 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 is 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. 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 6-20.

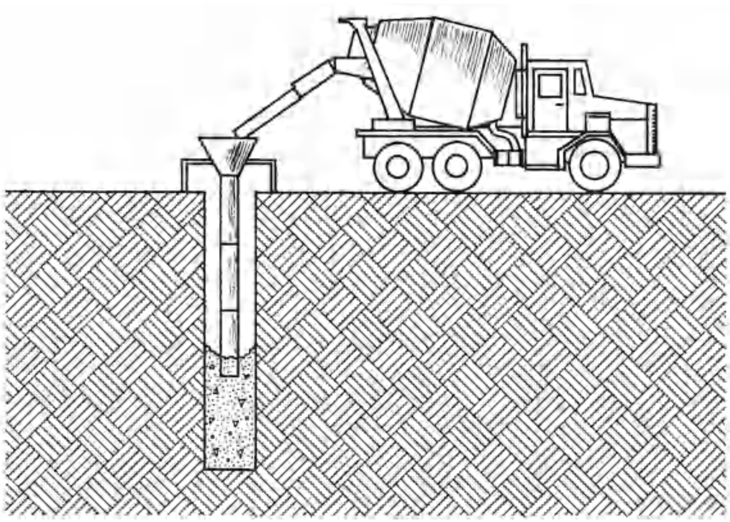


FIGURE 6-20. The use of a tremie for depositing concrete under water.

6-17. SAMPLE PROBLEMS

Example 6-1

Required: To determine the design strength of a longleaf southern pine timber pile driven into clay. Refer to Figure 6-21. Assume that the pile has been steam conditioned.

1. Load carrying capacity of pile due solely to axial compression.

It is assumed that load transfer to the soil is made through a combination of friction and end bearing. The critical section, therefore, may be taken at the butt.

From Formula (6-1) and Table 6-1:

$$C = \frac{4321 - 1.645 \times 707}{1.88} = 1{,}680 \text{ psi}$$

This stress shall be modified by length and by conditioning in accordance with the notes relating to Formula (6-1):

$$C_{\text{modified}} = 1680\,(1 + 0.002 \times 40)\,0.85 = 1{,}542 \text{ psi}$$

$$P_{\text{design}} = \frac{\pi 12^2}{4}\,(1542) = 174{,}396\ \# = 174 \text{ k} = 87 \text{ tons}$$

Note: In order to compute the transfer force for the pile in shear and end bearing, the following constants have been interpolated from Tables 6-3 and 6-5:

Location	Consistency	N	λ	c	N_c
Upper zone	medium	5	0.9	625	—
Lower zone	medium-stiff	12	0.4	1570	8

Also required are the following pile diameters:

$$\text{Diameter 22 feet below butt} = \frac{18}{40} \times 4 + 8 = 9.8 \text{ inches} = 0.817 \text{ ft}$$

$$\text{Average diam. in upper zone} = \frac{12.0 + 9.8}{2} = 10.9 \text{ inches} = 0.908 \text{ ft}$$

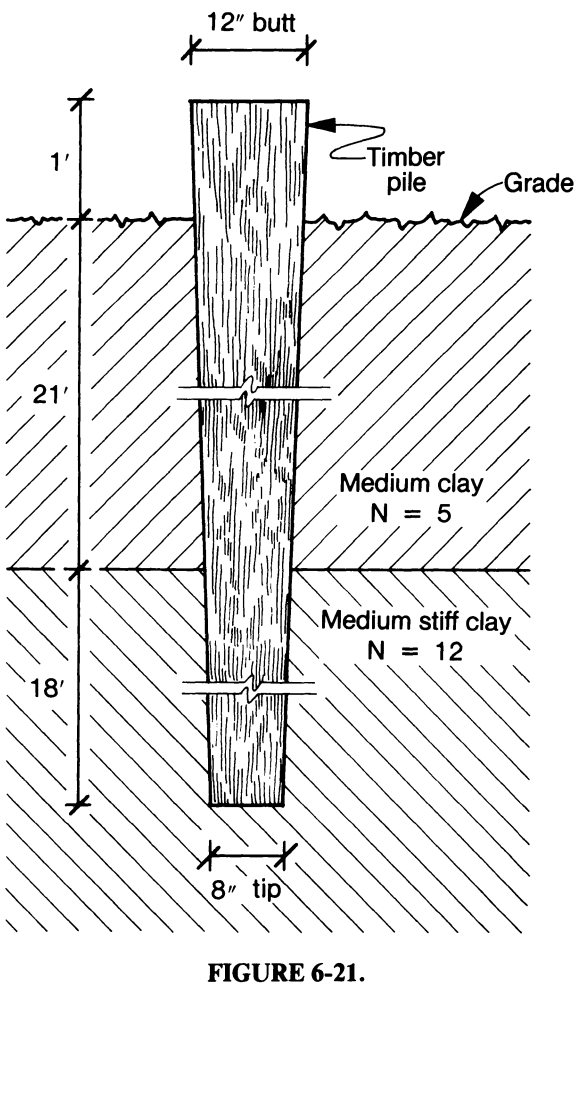


FIGURE 6-21.

$$\text{Average diam. in lower zone} = \frac{9.8 + 8.0}{2} = 8.9 \text{ inches} = 0.742 \text{ ft}$$

2. Shear resistance developed by cohesion, Formula (6-5):

Upper zone:

$$f = 0.9 \times 625 = 562 \text{ psf}$$
$$P_{\text{shear}} = 562\ \pi\ 0.908 \times 21 = 33{,}666\ \#$$

Lower zone:

$$f = 0.4 \times 1570 = 628 \text{ psf}$$
$$P_{\text{shear}} = 628\ \pi\ 0.742 \times 18 = 26{,}350\ \#$$
$$\text{Total resistance due to shear} = 60{,}016\ \#$$

3. Resistance developed by end bearing, Formula (6-7):

$$q_d = 1570 \times 8 = 12{,}560 \text{ psf}$$
$$P_{\text{bearing}} = 12{,}560 \times \frac{\pi 0.667^2}{4} = 4{,}388\ \#$$

4. Total combined resistance, from Formula (6-4):

$$P_{\text{ultimate}} = 60{,}016 + 4{,}388 = 64{,}404\ \#$$

5. A safety factor of 3 is recommended, particularly when the computed loads have not been substantiated by load tests, therefore:

$$P_{\text{design}} = \frac{64404}{3} = 21{,}468\ \# = 11 \text{ tons}$$

Note: The allowable transfer through shear and end bearing is considerably less than would be allowed due solely to axial compression. This is generally the case.

Example 6-2

Required: To determine the design strength of a 14 inch diameter concrete pier, drilled into clay with a continuous flight auger. Assume an ultimate compression strength of 2,500 psi. Soil conditions are as shown in Figure 6-22.

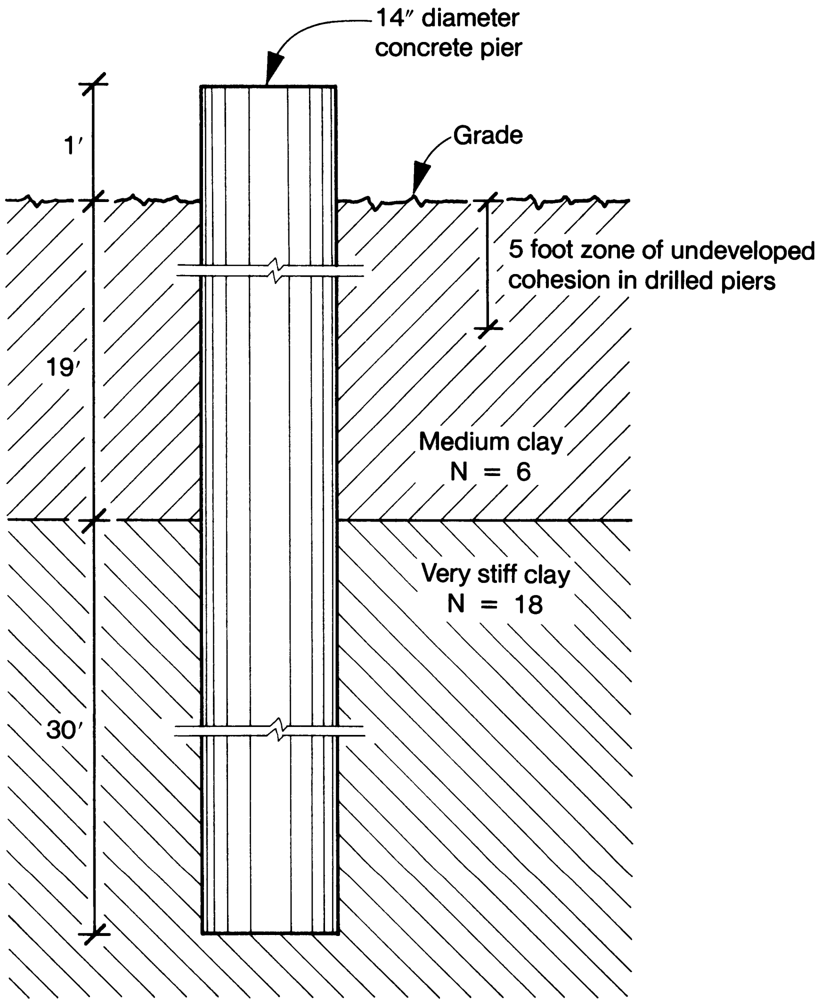


FIGURE 6-22.

1. Load carrying capacity of pier due solely to axial compression:

$$P_{\text{design}} = 0.33f'_c A_c = 0.33 \times 2500 \times \frac{\pi 14^2}{4} = 127{,}000\ \#$$

Note: In order to compute the transfer force for the pier in shear and end bearing, the following constants have been interpolated from Tables 6-3 and 6-5:

Location	Consistency	N	λ	c	N_c
Upper zone	medium	6	0.5	750	—
Lower zone	very stiff	18	0.5	2400	9

2. Shear resistance developed by cohesion, Formula (6-5):

Upper zone:

$$f = 0.5 \times 750 = 375 \text{ psf}$$

$$P_{\text{shear}} = 375\ \pi\ 1.167 \times (19 - 5) = 19{,}248\ \#$$

Lower zone:

$$f = 0.5 \times 2400 = 1{,}200 \text{ psf}$$

$$P_{\text{shear}} = 1200\ \pi\ 1.167 \times 30 = 131{,}984\ \#$$

$$\text{Total resistance due to shear} = 151{,}232\ \#$$

3. Resistance developed by end bearing, Formula (6-7):

$$q_d = 2400 \times 9 = 21{,}600 \text{ psf}$$

$$P_{\text{bearing}} = 21600 \times \frac{\pi 1.167^2}{4} = 23{,}104\ \#$$

4. Total combined resistance, from Formula (6-4):

$$P_{\text{ultimate}} = 151232 + 23104 = 174{,}336\ \#$$

5. For the reasons explained in Example 6-1, a safety factor of 3 is recommended.

$$P_{\text{design}} = \frac{174336}{3} = 58{,}112\ \# = 29 \text{ tons}$$

Example 6-3

Required: To determine the design strength of a 12 HP 53 steel pile, 50 feet long, and driven into medium sand, as indicated in Figure 6-23. The pile is a friction pile—end bearing is not to be considered.

1. Load carrying capacity of the column due solely to axial compression.

Assuming $F_y = 36.0$ ksi, and using a cross sectional area of 15.5, then:

$$P_{\text{design}} = 0.35 \times 36.0 \times 15.5 = 195.3 \text{ kips} = 98 \text{ tons}$$

2. The unit frictional resistance will be computed in accordance with Formula (6-6).

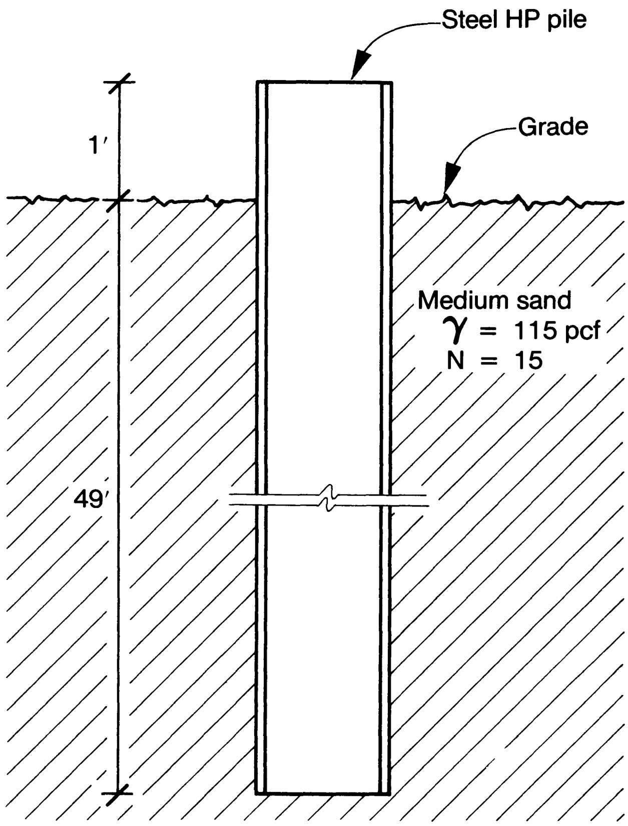


FIGURE 6-23.

The following information is required: From Section 6-7, the limiting value of vertical unit pressure is:

$$\gamma D_f = 115 \times 15 \times 1 = 1725 \text{ psf}$$

From Article 6-7:

$$K = 0.75, \text{ assumed for medium sand.}$$

From Table 6-4:

$$\tan \delta = 0.4,$$

assuming the steel to be somewhat rough and rusted. From Table 6-2: Pile perimeter is 4.00 feet.

For upper zone, 15 feet:

$$f = \frac{1725}{2} \times 0.75 \times 0.4 = 259 \text{ psf}$$

$$P_{\text{shear}} = 259 \times 15 \times 4 = 15{,}540 \ \#$$

For lower zone, 34 feet:

$$f = 1725 \times 0.75 \times 0.4 = 518 \text{ psf}$$

$$P_{\text{shear}} = 518 \times 34 \times 4 = 70{,}448 \ \#$$

$$\text{Total resistance from shear} = 85{,}988 \ \#$$

3. With a safety factor of 3:

$$P_{\text{design}} = \frac{85988}{3} = 28662 \ \# = 14 \text{ tons}$$

Example 6-4

Required: To determine the design strength of a concrete filled steel shell pile driven into sand, and as detailed in Figure 6-24.

The following information has been given:

Steel shell: 12″ OD, 9 gage steel with $t = 0.1495''$, $\dot{F}_y = 50.0$
Concrete fill: 4000 psi, $E = 3{,}640{,}000$ psi
Upper sand layer: $\gamma = 112$ pcf, $N = 16$, $\phi = 31°$
Lower sand layer: $\gamma = 124$ psf, $N = 34$, $\phi = 36°$

1. Load carrying capacity of pile due solely to axial compression.

$$\text{Area of concrete } A_c = \frac{\pi (12 - 2 \times 0.1495)^2}{4} = 107.53 \text{ sq. in.}$$

$$\text{Area of steel } A_s = \frac{\pi (12)^2}{4} - 107.53 = 5.57 \text{ sq. in.}$$

From Formula (6-2):

$$\begin{aligned} P_{\text{design}} &= 0.35 \times 50{,}000 \times 5.57 + 0.33 \times 4{,}000 \times 107.53 \\ &= 97{,}475 + 141{,}940 = 239{,}415 \ \# = 120 \text{ tons} \end{aligned}$$

But according to Formula (6-3), the total load must be distributed between the steel and the concrete in such proportion as to insure that the deformation of each

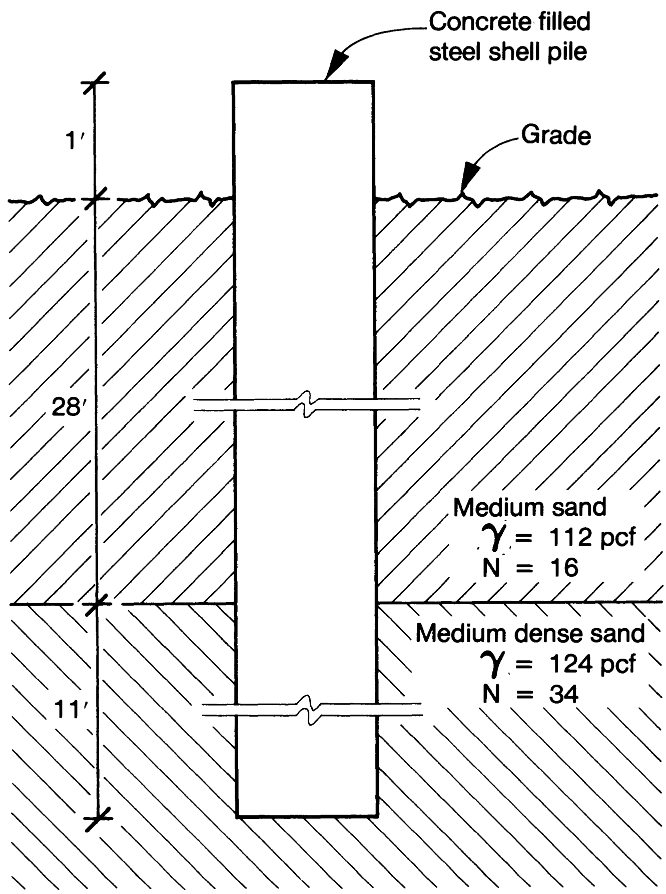


FIGURE 6-24.

material will be equal. Therefore:

$$\Delta L = \frac{P_s}{5.57 \times 29.0} = \frac{P_c}{107.53 \times 3.64}$$

(*Note:* $E_c = 3.64$.) From which:

$$2.423\ P_s = P_c$$

If the load carried by the concrete is 141,940, as being the limiting value determined from Formula (6-2), then the load carried by the steel is limited to 58,580, as determined from equal shortening. Then:

$$P_{\text{design}} = 58{,}580 + 141{,}940 = 200{,}520\ \# = 200\ \text{k} = 100\ \text{tons}$$

The resolution of this discrepancy between code and performance is left to the reader.

2. The shear developed by skin friction is computed for each layer of soil according to Formula (6-6). The following data has been interpolated from that given in Article 6-7:

Layer	K	Tan δ[a]
Upper	0.7	0.60
Lower	0.9	0.73

[a] For corrugated metal use tan ϕ.

The maximum amount of vertical unit pressure, according to Formula (6-6) is:

$$\gamma D_f = 112\,(15 \times 1) = 1680 \text{ psf}$$

For the upper layer:

$$f = 1680 \times 0.7 \times 0.60 = 706 \text{ psf}$$

$$P_{\text{shear}} = \frac{706}{2} \times \pi \times 1 \times 15 + 706 \times \pi \times 1 \times 13 = 45{,}468 \text{ \#}$$

For the lower layer:

$$f = 1680 \times 0.9 \times 0.73 = 1{,}104 \text{ psf}$$

$$P_{\text{shear}} = 1104 \times \pi \times 1 \times 11 = 38{,}151 \text{ \#}$$

$$\text{Total ultimate resistance due to shear} = 83{,}619 \text{ \#}$$

3. The ultimate stress that can be developed in end bearing is computed from Formula (6-9). Using $N_q = 38$, from Figure 4-2:

$$q_d = 1680 \times 38 = 63{,}840 \text{ psf}$$

$$P_{\text{bearing}} = 63840 \times \frac{\pi \times 1^2}{4} = 50{,}140 \text{ \#}$$

4. Total combined resistance, from Formula (6-4) is:

$$P_{\text{ultimate}} = 83619 + 50140 = 133{,}759 \text{ \#}$$

5. Using a safety factor of 3, the design capacity of the pile is:

$$P_{\text{design}} = \frac{133759}{3} = 44{,}586 \text{ \#} = 22 \text{ tons}$$

Example 6-5

Required: To determine the design load of a 16″ diameter concrete pier, installed into sandy soil with a hollow shaft auger. Refer to Figure 6-25.

The following information has been given:

Concrete strength is 4000 psi.
Upper sand layer: $\gamma = 95$ pcf, N = 8, $\phi = 29°$
Lower sand layer: $\gamma = 121$ psf, N = 30, $\phi = 35°$

1. Load carrying capacity of pile due solely to axial compression.

$$P_{design} = 0.33 \times 4000 \times \frac{\pi\ 16^2}{4} = 265{,}400\ \# = 133 \text{ tons}$$

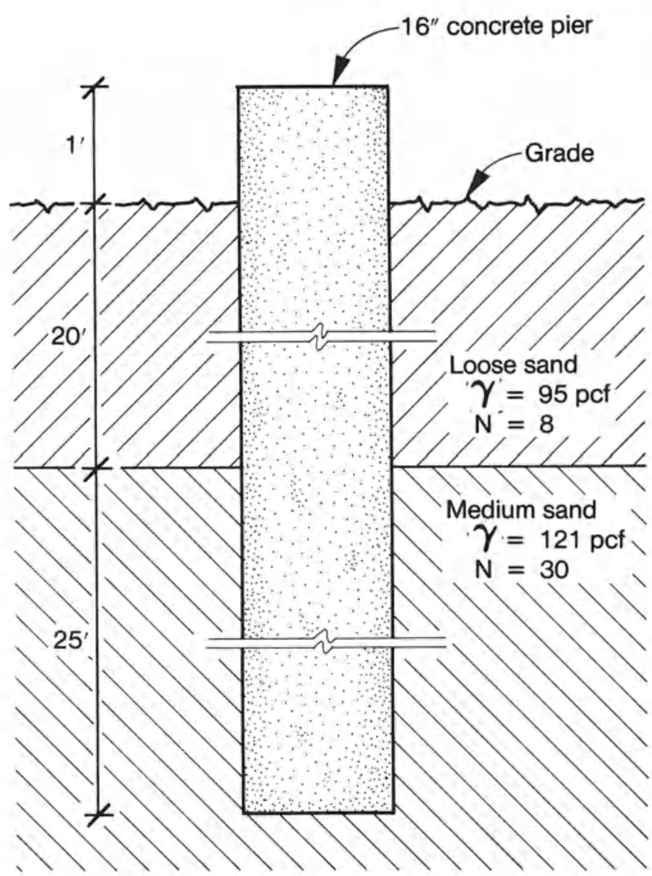


FIGURE 6-25.

2. The shear developed by skin friction is computed for each layer of soil according to Formula (6-6). The following data has been interpolated from that given in Article 6-7:

Layer	K[a]	Tan δ
Upper	0.52	0.45
Lower	0.43	0.45

[a] For piers use $K = 1 - \sin \phi$

The maximum amount of vertical unit pressure, according to Formula (6-6) is:

$$\gamma D_f = 95\ (15 \times 1.33) = 1895 \text{ psf}$$

For the upper layer:

$$f = 1895 \times 0.52 \times 0.45 = 443 \text{ psf}$$

Note: In this particular instance, the 15 diameter limitation occurs at the change in layers.

$$P_{\text{shear}} = \frac{443}{2} \times \pi \times 1.33 \times 20 = 18{,}510 \ \#$$

For the lower layer:

$$f = 1895 \times 0.43 \times 0.45 = 367 \text{ psf}$$

$$P_{\text{shear}} = 367 \times \pi \times 1.33 \times 25 = 38{,}336 \ \#$$

Total ultimate resistance due to shear = 56,846 #

3. The ultimate stress that can be developed in end bearing is computed from Formula (6-9): Using $N_q = 33$, from Figure 4-2:

$$q_d = 1895 \times 33 = 62{,}535 \text{ psf}$$

$$P_{\text{bearing}} = 62535 \times \frac{\pi \times 1.33^2}{4} = 86{,}879 \ \#$$

4. Total combined resistance, from Formula (6-4) is:

$$P_{\text{ultimate}} = 56846 + 86879 = 143{,}725 \ \#$$

5. Using a safety factor of 3, the design capacity of the pile is:

$$P_{\text{design}} = \frac{143725}{3} = 47{,}908 \ \# = 24 \text{ tons}$$

Example 6-6

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. Assume 3000 psi concrete.

The required shaft area may be computed as follows:

$$0.33 \times 3000 = \frac{1{,}800{,}000}{\text{Area}}$$

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 can be considered to be adequate.

The dead weight of the caisson is:

$$\frac{\pi 4^2}{4} \times 50 \times 150 = 94{,}247\ \#$$

The required bell area may be computed as follows:

$$q_a = 40 \times 2 = \frac{1800 + 94}{\text{Area}}$$

Therefore, the required area is 23.7 square feet, and the corresponding diameter is 5.49 feet. Bells are usually specified in 6″ increments, therefore, a 6′-6″ bell should be used.

7

Lateral Earth Pressure

7-1. GENERAL

The purpose of a retaining wall is to provide for an abrupt change in grade as required by architectural or engineering considerations. In order for this to be accomplished 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.

7-2. THE CONCEPT OF LATERAL EARTH PRESSURE

The concept of lateral earth pressure can be explained by an understanding of the behavior of a soil mass when it is restrained against lateral movement or when it is free to move. In order to discuss this behavior, several theoretical properties of the soil must be defined.

Angle of Repose θ

The angle of repose can best be explained by performing a relatively simple experiment in which a quantity of cohesionless soil is allowed to fall freely onto 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 the amount is so small that it is customarily ignored. The angle that

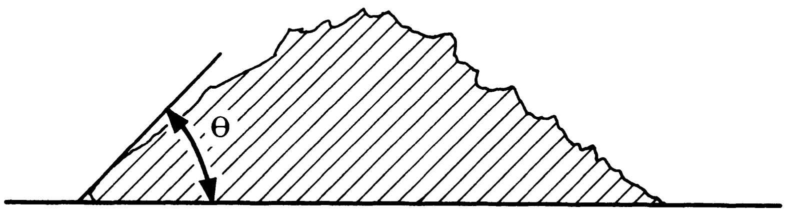


FIGURE 7-1. Profile through a freely deposited soil.

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 7-1.

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, while fine grained soils such as silt and clay develop their resistance through a property called cohesion. This is described in Section 7-3, in which the Coulomb equation for shear resistance is discussed.

Angle of Rupture α

When a mass of earth 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*. This plane is actually not a plane, but is slightly concave. The amount of out-of-line 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 can be demonstrated in Figure 7-2, 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 slide down and fall out of the box. The movement of this earth wedge upon the

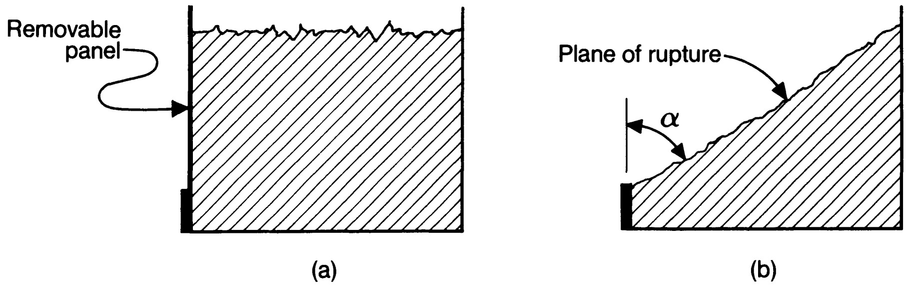


FIGURE 7-2. Action of an earth mass when free to slide. (a) before release, (b) after release.

release of physical restraint is clear evidence of the existence 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 a 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 ϕ

The angle of internal friction, as introduced in Section 2-9, plays a significant role in the theory upon which the development of lateral earth pressure is based.

For angles representative of various general soil classifications refer to Table 2-3. 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 judgment dictates the selection of conservative values. The following rules of thumb may serve as guidelines in this selection:

1. 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 four general soil classifications as follows:

   gravel > sand > silt > clay

2. The pressure exerted by a backfill can be assumed to vary between the four general soil classifications as follows:

   hard clay > soft clay > dense granular soil > loose granular soil

It is also known that the angle of internal friction decreases slightly as moisture content is increased. Since all in-place backfill will contain moisture, it is recommended that the lower values be chosen when using that table.

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 is illustrated in Figure 2-4.

7-3. THE COULOMB EQUATION FOR SHEAR RESISTANCE

All soils have the ability to develop resistance to shear. Different kinds of soil groups develop this resistance in different ways.

1. In sands and gravels this resistance is due to the physical interlocking of the soil particles. 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 sandpaper 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 a combination of the separate resistances of friction and cohesion.

The 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:

$$s = c + p \tan \phi - \text{psf} \tag{7-1}$$

Where:

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

When the total resistance to shear over a given area is required, the unit resistance as computed from Formula (7-1) must be multiplied by the area involved. In the case of computations relative to total resistance, as indicated in Figure 7-5, the unit resistance must be multiplied by an area equal to the length of the assumed

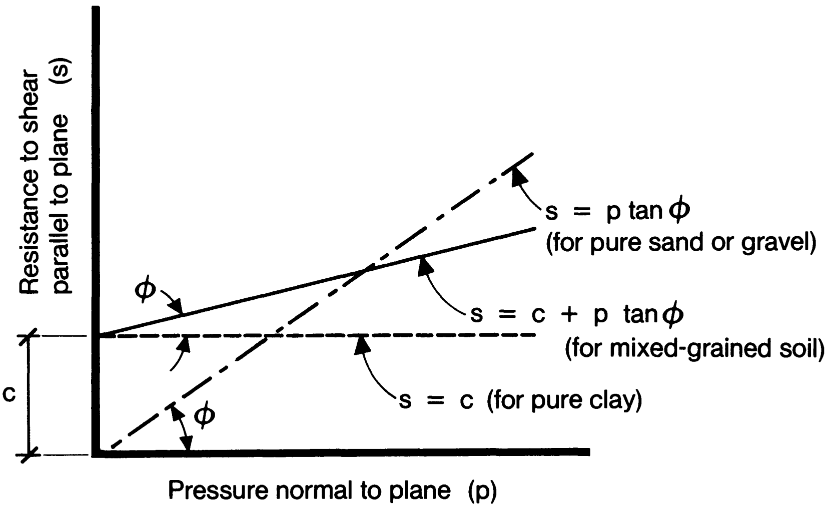


FIGURE 7-3. Graphic illustration of the Coulomb equation for shear resistance of different kinds of soil.

plane of rupture times one foot measured along the length of the wall. The total resistance then, will be given in terms of pounds per linear foot of wall.

The resistance to shear as determined by the Coulomb equation can be illustrated graphically, as shown in Figure 7-3.

7-4. THE CONCEPT OF ACTIVE EARTH PRESSURE

It has been shown that soil can be deposited in a pile no higher than that of the free formed pile unless it is deposited against a restraint, such as a retaining wall. It follows, then, that when the soil is deposited against the 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 their characteristics as to angle of repose, angle of rupture, and angle of internal friction, all of which will ultimately affect the magnitude of the active pressure exerted by the soil mass against the wall.

The soil mass is shaped like a wedge, the geometry of which is illustrated in Figure 7-4. Remember, the plane of rupture is actually slightly dished, but for the purpose of calculations is assumed to be straight.

When the weight of the earth plus any surcharge produces forces which overcome the shearing resistance of the earth, then the earth wedge will slide imperceptibly down along the plane of rupture, thus producing active pressure against 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.

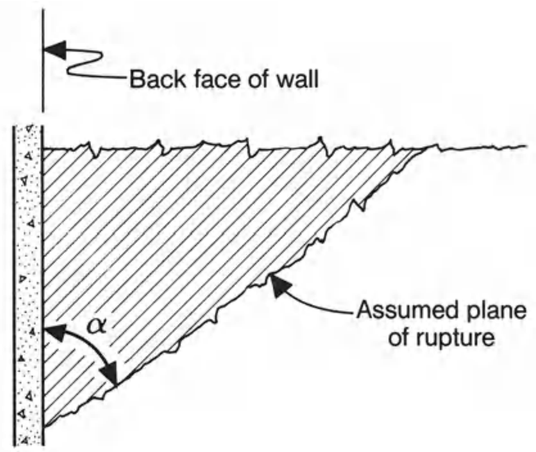


FIGURE 7-4. The development of lateral pressure due to a tendency for a wedge of soil to slide down along a potential plane of rupture.

7-5. THE WEDGE THEORY OF ACTIVE EARTH PRESSURE

The wedge theory of active earth pressure developed in this section is based upon the following assumptions, of which numbers 2 and 3 are conservative, and, therefore, on the side of safety:

1. The earth behind the wall is free of fines and has no measureable cohesion. Therefore:

$$c = 0 \quad \text{and} \quad s = p \tan \phi$$

2. The stabilizing effect of the earth in front of the wall is ignored in all computations.
3. The back 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 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.
7. The earth behind the wall is reasonably uniform, and the angle of internal friction is reasonably constant.

The analysis is started by isolating the earth wedge as a free body diagram, and by indicating all the forces which act upon it. Equilibrium equations are then developed for the components of all forces acting vertically and horizontally on the wedge. The end result of this analysis is to determine the lateral pressure for

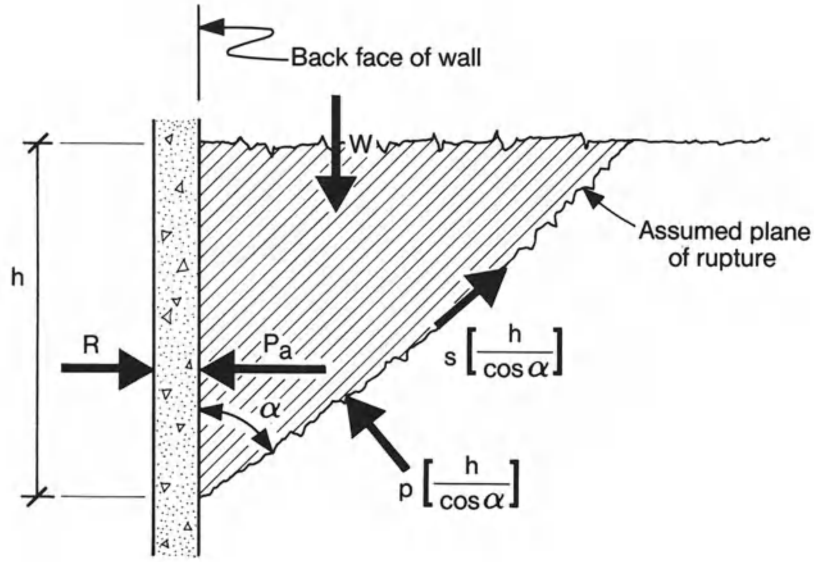


FIGURE 7-5. Free body diagram of the earth wedge on the verge of failure.

which the wall must be designed. The free body diagram referred to is shown in Figure 7-5.

In Figure 7-5:

$W = \frac{1}{2}\gamma h (h \tan \alpha)$ = the weight of soil comprising the earth wedge

$p\left(\frac{h}{\cos\alpha}\right)$ = the total force acting normal to the plane of rupture

$s\left[\frac{h}{\cos\alpha}\right] = p \tan\phi \left[\frac{h}{\cos\alpha}\right]$ = the total frictional resistance of the soil, acting parallel to the line of rupture

P_a = the horizontal force exerted on the wall by the sliding action of the earth wedge

R = the horizontal reaction of the wall to the force P_a

By taking the summation of the forces acting vertically on the wedge:

$$W = p\left(\frac{h}{\cos\alpha}\right)\sin\alpha + p\tan\phi\left(\frac{h}{\cos\alpha}\right)\cos\alpha$$

from which:

$$W = ph\tan\alpha + ph\tan\phi$$

By taking the summation of the forces acting horizontally on the wedge:

$$R = P_a = p\left(\frac{h}{\cos\alpha}\right)\cos\alpha - p\tan\phi\left(\frac{h}{\cos\alpha}\right)\sin\alpha$$

from which:

$$P_a = ph - ph\tan\phi\tan\alpha$$

By solving each equation for ph, then 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:

$$P_a = \left(\frac{\gamma h^2\tan\alpha}{2}\right)\times\left(\frac{1-\tan\phi\tan\alpha}{\tan\phi+\tan\alpha}\right) \tag{7-2}$$

7-6. COEFFICIENT OF ACTIVE PRESSURE

The variables ϕ and α are functions which are solely dependent on the characteristics of the particular soil under consideration. The preceding formula for the active pressure for that particular soil can then be rewritten as follows:

$$P_a = \frac{1}{2}K_a\gamma h^2 \tag{7-3}$$

where

$$K_a = \frac{1-\tan\phi\tan\alpha}{\dfrac{\tan\phi}{\tan\alpha}+1} \tag{7-4}$$

K_a is called the *coefficient of active pressure.*

The design of a retaining wall must be based on the maximum amount of thrust to which it can be subjected. This thrust is that which 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. This coefficient depends solely upon the angle of internal friction and the assumed angle of rupture. For a given soil, since the angle of internal friction is constant, the coefficient depends only on the angle of rupture. The maximum value of this coefficient can be found, therefore, by differentiating Equation (7-4) with respect to the angle of rupture, setting the derivative equal to zero, and solving. This work establishes that the following angle of rupture will produce the maximum value of the coefficient:

$$\tan \alpha = \sec \phi - \tan \phi = \frac{1 - \sin \phi}{\cos \phi} \tag{7-5}$$

The maximum value of the coefficient of active pressure can now be found by substituting the above value of $\tan \alpha$ into Equation (7-4), and solving. This results in the following:

$$K_a = (\sec \phi - \tan \phi)^2 \tag{7-6}$$

or

$$K_a = \frac{1 - \sin \phi}{1 + \sin \phi} \tag{7-7}$$

or

$$K_a = \tan^2 \left(45° - \frac{\phi}{2}\right) \tag{7-8}$$

All three of the above forms of the equation are in general usage. The latter form is preferred by the author and will be used in the remainder of this text.

Formulas (7-5) and (7-8) 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 7-1.

TABLE 7-1. Correlation between Angle of Internal Friction, Angle of Rupture, and 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

7-7. 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 (7-8) and (7-3) can be used to accurately determine the numerical value of the total horizontal force for which a retaining wall must be designed, subject only to the requirement that the work as built conforms to the assumptions under which the equations were developed, as itemized in Section 7-5.

7-8. EQUIVALENT LIQUID PRESSURE THEORY

Formula (7-3), which numerically identifies the horizontal force P_a, can be rewritten in the following form:

$$P_a = \frac{1}{2}(K_a \gamma h)\, h \tag{7-9}$$

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 called the *equivalent liquid pressure theory.* The examples enumerated in Table 7-2 illustrate the use of this theory for the more frequently encountered conditions of loading.

For pressure diagrams representative of the four generalized loading types refer to Figures 7-6 to 7-9.

TABLE 7-2. Common Types of Active Pressure.

Type	Earth Surface	Surcharge	Water Table	Figure No.
1	level	no	no	7-6
2	level	yes	no	7-7
3	level	no	yes	7-8
4	sloped	no	no	7-9

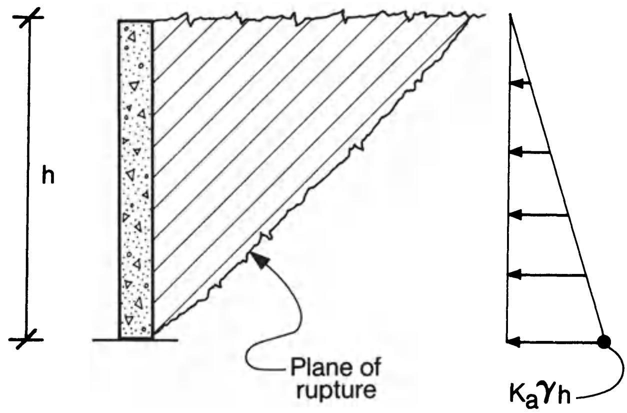


FIGURE 7-6. Pressure diagram produced solely by earth.

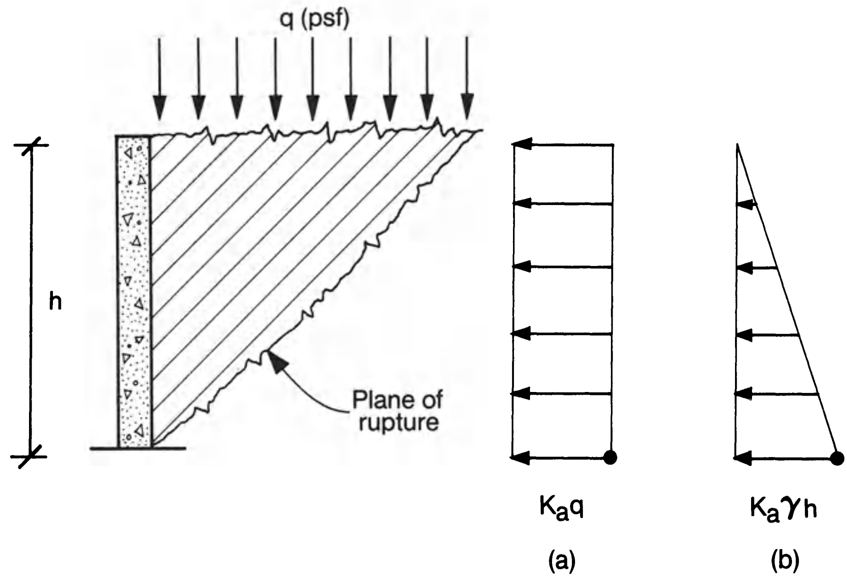


FIGURE 7-7. Pressure diagrams produced by surcharge (a), and earth (b).

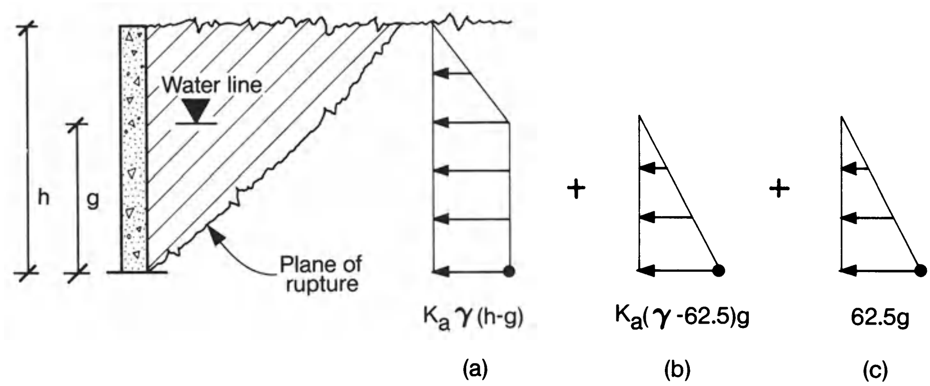


FIGURE 7-8. Pressure diagrams produced by earth (a) and (b), and ground water (c).

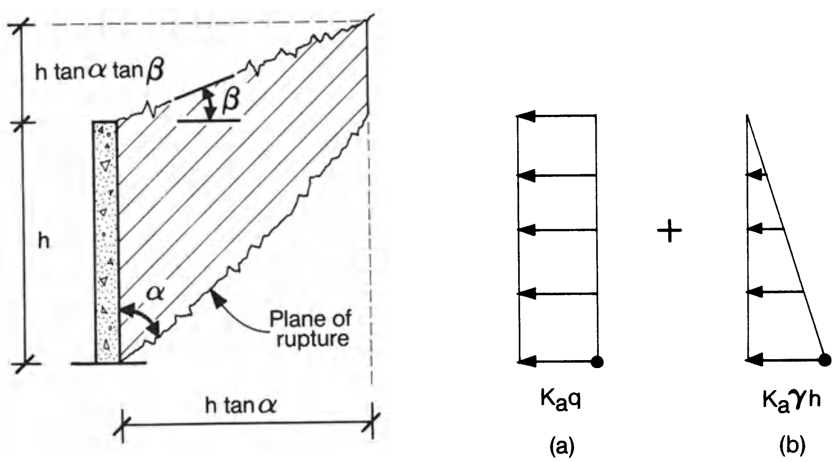


FIGURE 7-9. Pressure diagrams produced by sloping earth, where (a) is due to the earth above the wall (acting as a surcharge), and (b) is due to the earth beside the wall. The surcharge $q = 0.5\gamma$ (h tan α tan β).

7-9. NUMERICAL ACCURACY OF K_a AND P_a

The validity of all computations involving the equivalent liquid pressure theory depend upon the accuracy of the numerical values of the following two things:

1. The coefficient of active pressure K_a: This coefficient, as shown in Section 7-6, is solely dependent upon the magnitude of the angle of internal friction of the soil which the wall is to support. Values for K_a based upon this

dependency have previously been given in Table 7-1. The magnitude of this angle, and therefore the value of K_a, can be approximated from the borings because of a correlation that exists between it and the blow count N, as recorded during the standard penetration test. This correlation is indicated in Figure 2-4 of Chapter 2.

2. The horizontal force P_a exerted on the wall by the sliding action of the earth wedge: 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 Table 2-1 of Chapter 2 and Table 10-3 of Chapter 10.

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.

It must be recognized that in almost all areas of soil mechanics there are unknowns and intangibles and inconsistencies which 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 of Chapter 3. 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.

7-10. CHARTS FOR ESTIMATING BACKFILL PRESSURE

Reasonable estimates of the earth pressure acting on a wall can be made by use of the charts included in this section. 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 about 20 feet.

In order to use these charts the material that will be used for backfill must first be classified in one of the four 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.
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.

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 is the horizontal pressure, and
γ is 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 to take 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 which bears on the footing must lift. In order for this earth to lift it must shear away from the earth situated beyond the footing. This shear, then, 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, and is given analytically in terms of the coefficient K_v.

It is left to the individual designer as to whether he will incorporate the restraining effect of this shear into his computations when designing walls in accordance with these charts. Since most engineers tend to be conservative when dealing with soils, the author recommends that this shear be ignored in all retaining wall computations.

Numerical values for the coefficients K_h and K_v may be taken directly from the charts given in Figure 7-10.

7-11. SAMPLE PROBLEMS

Note: When dealing with lateral pressure for the investigation or design of walls subjected to lateral earth pressure, pressures, loads, forces, shears, and moments are computed on the basis of 1 linear foot of wall.

Example 7-1

Required: To determine the pressure gradient and total force acting against a 14 foot high wall, subject to the following:

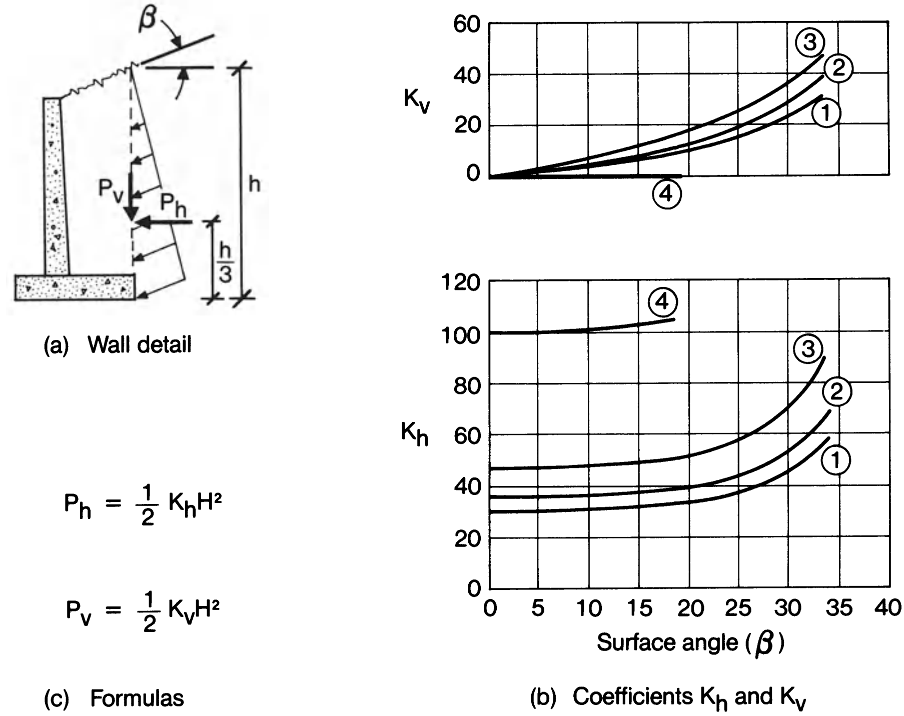


FIGURE 7-10. Charts for estimating backfill pressure. [Ref. 16]

From laboratory analysis:

$$\gamma = 110 \text{ pcf} \quad \text{and} \quad \phi = 32°$$

From Table 7-1:

$$K_a = 0.31$$

Refer to Figure 7-6 for the method, and to Figure 7-11 for the solution.

Example 7-2

Required: Repeat Example 7-1, but add a surcharge of 400 psf.

Refer to Figure 7-7 for the method, and to Figure 7-12 for the solution.

Example 7-3

Required: Repeat Example 7-1, but add a water table 6 foot up from base.

Refer to Figure 7-8 for the method, and to Figure 7-13 for the solution.

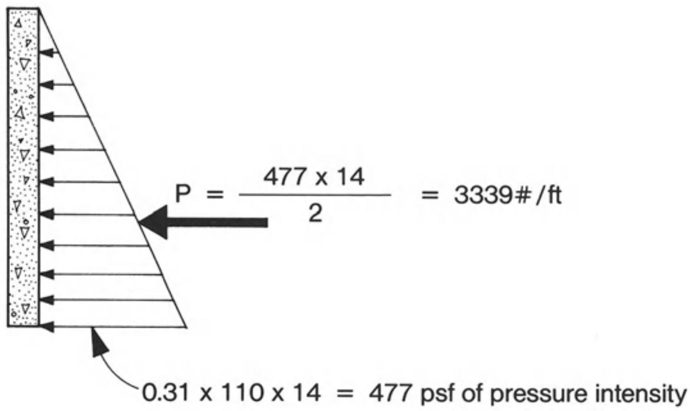


FIGURE 7-11.

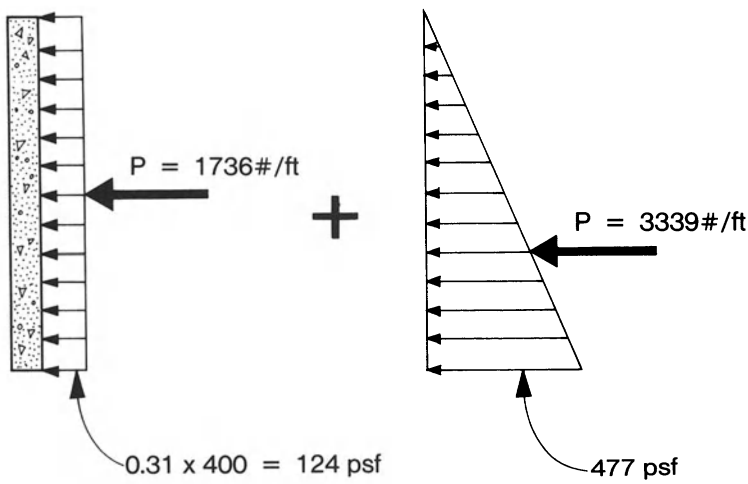


FIGURE 7-12.

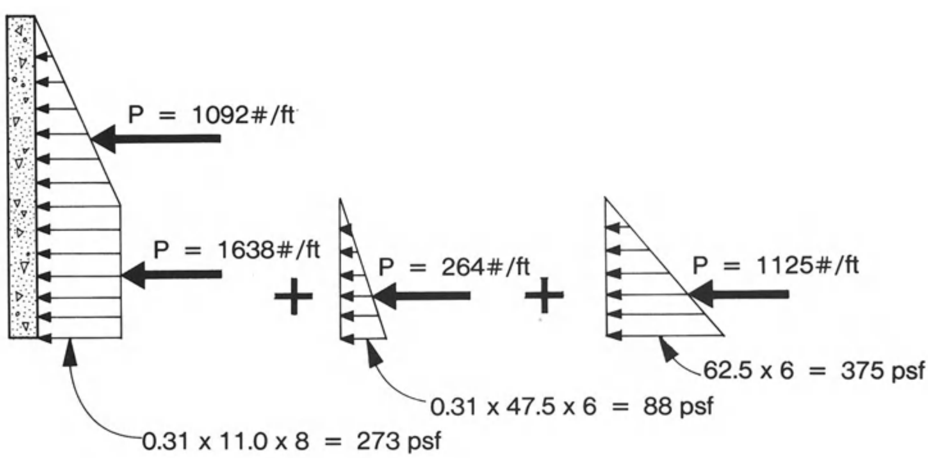


FIGURE 7-13.

Example 7-4

Required: Repeat Example 7-1, but slope the surface of the backfill 30°.

From Table 7-1:

$$\alpha = 29°$$

Refer to Figure 7-9 for the method, and to Figure 7-14 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:

$$q = \frac{1}{2}\, 110 \times 14 \times \tan 29° \tan 30° = 246 \text{ psf}$$

Example 7-5

Required: To determine the total force acting against a 14 foot high wall, by using the charts for estimating backfill pressure.

Assume the surface of the backfill to be level. Also assume the backfill to be type 2.

Refer to Figure 7-10 for the method, and to Figure 7-15 for the solution.

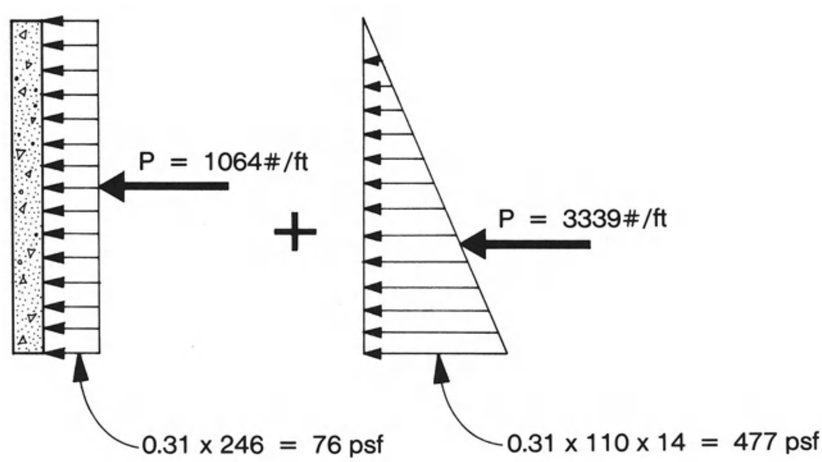


FIGURE 7-14.

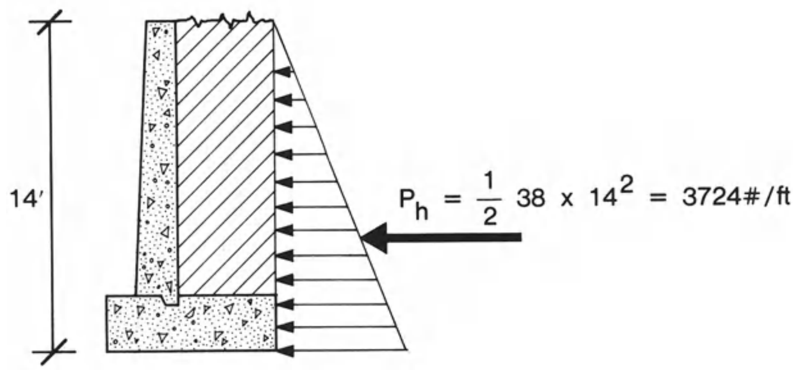


FIGURE 7-15.

Example 7-6

Required: Repeat Example 7-5, but slope the surface of the backfill 30°.

Refer to Figure 7-10 for the method, and to Figure 7-16 for the solution.

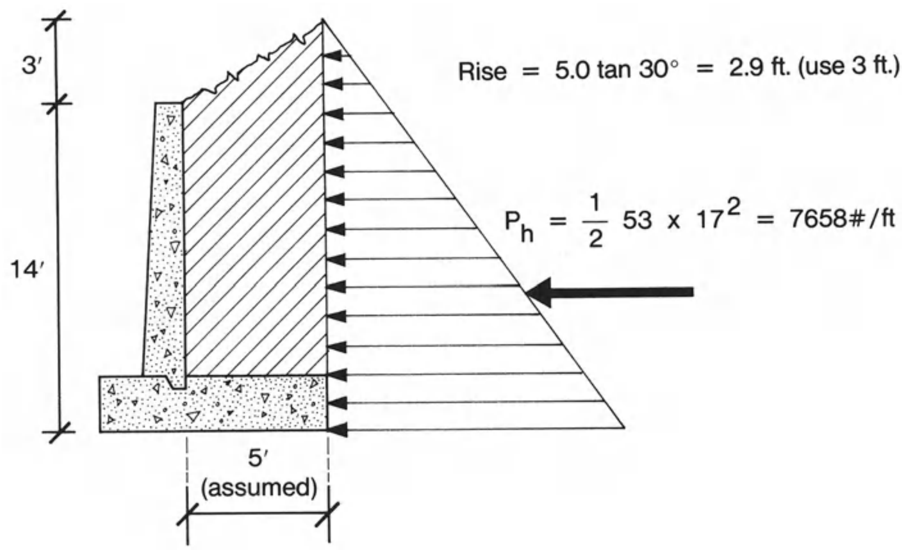


FIGURE 7-16.

8

Walls — Construction Details

8-1. 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 provide for abrupt changes in the contours of the existing grade. These changes may be of a temporary nature or they may be permanent. Examples of where such changes are required is as follows:

1. In the construction of highways, particularly at ramps, bridges and cross-overs
2. Where temporary working space is required in which to construct the underground areas of a building or other structure
3. At the exterior walls of basements of buildings and at interior areas such as elevator pits, pipe tunnels, and other mechanical spaces
4. As part of the architectural treatment of the overall development of the site
5. Where bulkheads are required for the control of water and land erosion in coastal areas

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.

8-2. 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, 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 8-1.

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 8-2. 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 turn-downs 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.

8-3. SHEET PILING RETAINING WALLS

A typical detail of a sheet piling wall is illustrated in Figure 8-3. There are two ways by which this type of wall can be built, depending upon 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. As the piling is advanced the earth on the front side 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. The use of piling in this latter case is illustrated in Figure 8-4.

Sheet piling is always ribbed for strength and must be designed not only for earth pressure but also for the dynamic forces induced by the driving operation. Wales are horizontal members placed hard against the outside face of the sheet piling. Their purpose is to support the piling and transfer the lateral pressure, by beam action, to the tiebacks. Wales are usually heavy timber, but could also be structural steel. Tiebacks are prestressed cables extending behind the wall where

FIGURE 8-1. Typical sliding failures of an earth embankment.

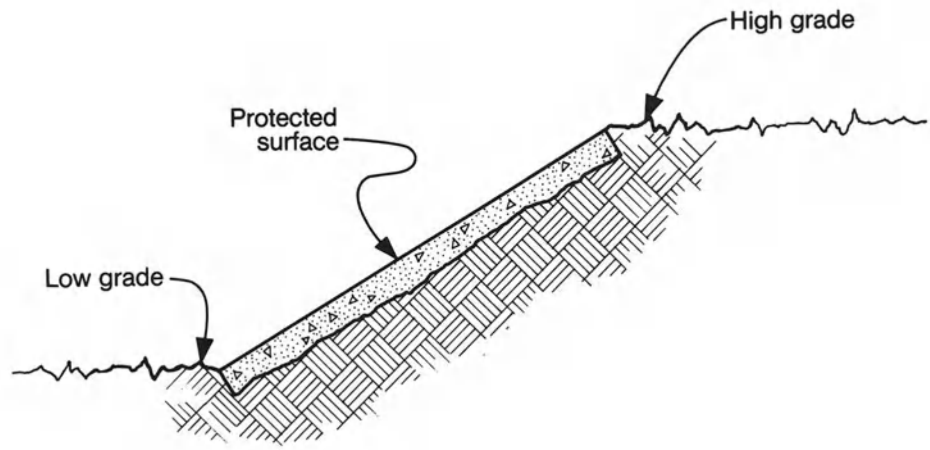


FIGURE 8-2. Earth embankment stabilized with a concrete cover.

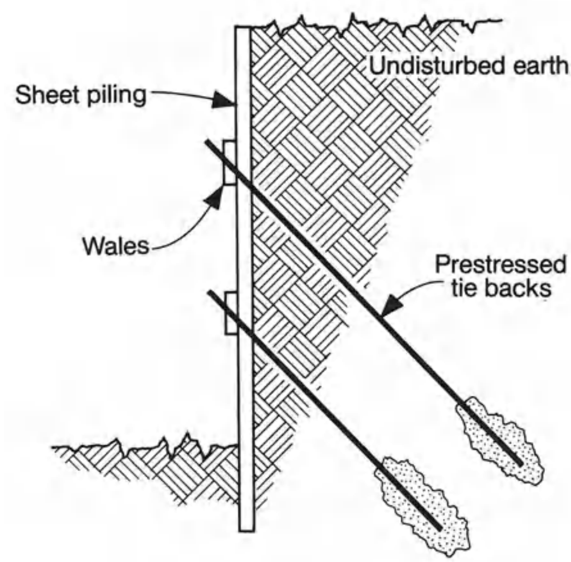


FIGURE 8-3. Typical detail of a steel sheet piling retaining wall.

FIGURE 8-4. Bethlehem PZ27 sheet piling stabilizes this busy roadway in a marshy location near Williamsburg, Va. [Ref. 3]

they are anchored into the underlying bedrock or undisturbed earth. For an in-depth discussion of prestressed cables refer to Section 8-10.

8-4. SOLDIER BEAM RETAINING WALLS

A typical detail of a soldier beam wall is illustrated in Figure 8-5. Soldier beam walls are used in situations where major changes in grade are required, and are usually considered to be temporary. A prime example of the use of this type of wall is where deep excavation is required for the installation 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 to W sections except that their flanges and web are of equal thickness. Soldier beams are placed approximately 8 feet on centers 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, 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, cut to fit between the webs of the soldier beams, and placed hard against the inside face of the outside 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. Tiebacks are for the purpose of providing lateral support to the soldier beams, and are similar to those described in Section 8-3.

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 may be used. Such an installation is illustrated in Figure 8-6.

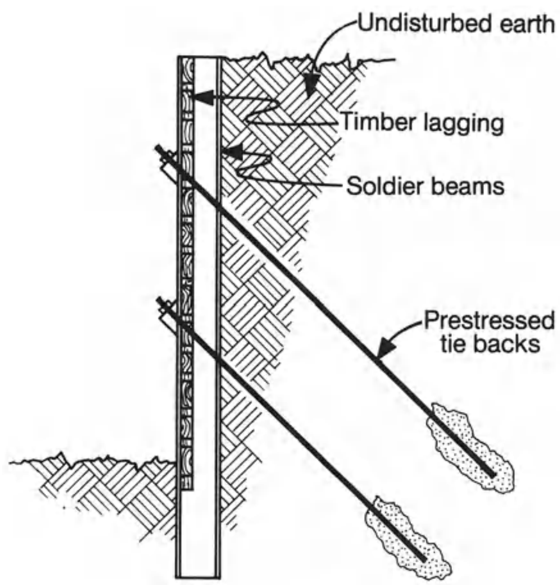


FIGURE 8-5. Typical detail of a soldier beam retaining wall.

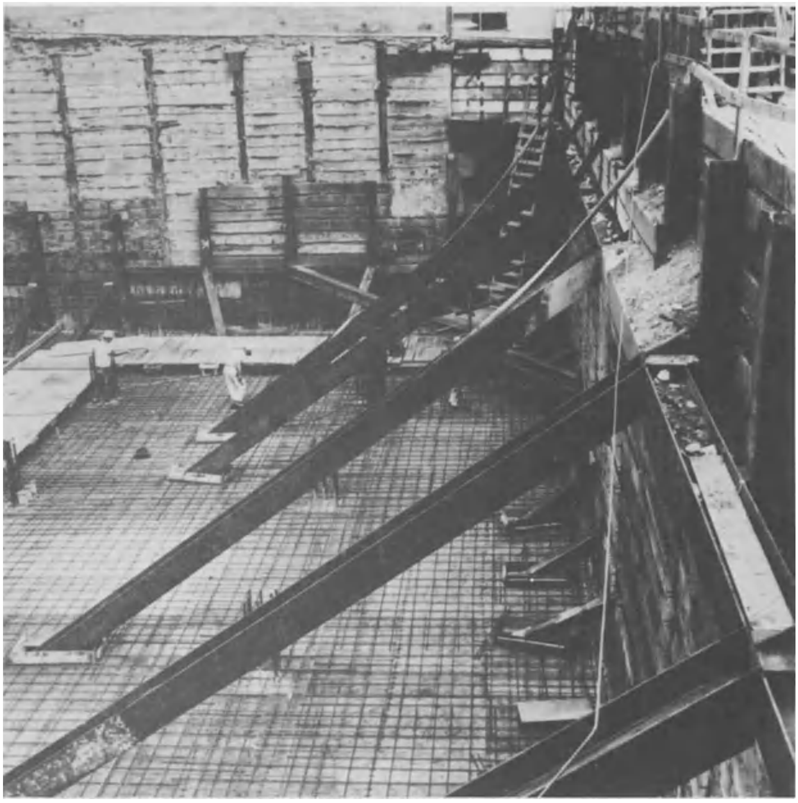


FIGURE 8-6. A soldier beam wall, stabilized with wales and inclined shores. [Ref. 3]

8-5. BASEMENT WALL—TYPICAL DETAIL

The purpose of a basement wall is to provide an enclosure for a usable space in a building when that space is to be 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 8-7. 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 from leakage of water and/or influx of moisture. This protection has not been shown on any of these details because 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. Considerable excavation, therefore, will be required. Heavy earth moving machinery will be used to bring the rough excavation down to the level of the subgrade below the basement slab. Care should be taken, however, to not extend the excavation below the top of the footing. Wood forms, called *screed rails* (described in Section 5-2) are then set to the exact elevation of the top of 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 formwork for the walls can be set directly on the footing and construction of the walls and other elements of the building can be started. Exterior backfill, however, can not be placed until the 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, then the wall must be temporarily supported, using one of the following methods:

1. Construct timber or steel shoring from the wall down to, and set into, the subgrade on the basement side of the wall.
2. Tie the wall back with prestressed cables extending back into the undisturbed earth on the exterior side of the wall.

Both of these methods of temporary support are expensive and time consuming, and should only be used after alternatives have been carefully considered.

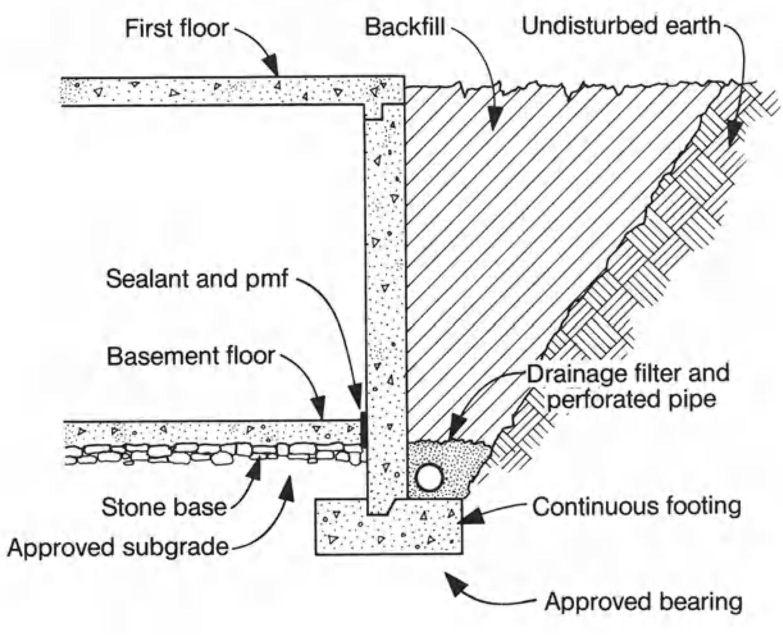


FIGURE 8-7. Typical detail of a basement wall.

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 3 to 4 feet below the slab on ground.

With reference to adequate support as provided by the soil, it is the opinion of the author that soils whose computed allowable bearing pressure is 1 tsf or less, should never be used to support any kind of building foundation, and that pressures less than 1.5 tsf should be highly suspect. Refer to Section 4-15.

The method of using a continuous wall footing becomes less and less practical as the footing must be lowered to find 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 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, then the grade beam can be designed to span between these columns. When building columns have not been placed within the wall then 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 centers along the wall, subject to the type of structural system being used.

8-6. BASEMENT WALL—SPECIAL CONDITIONS

Face Brick below Grade

The detail shown in Figure 8-8 provides for 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, which is commonly called a *brick shelf,* should be placed in coursing several courses below grade. (*Coursing* means that the shelf should line up with the top of a brick.) The shelf is normally specified to be 4½ to 5 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.

Exposed Concrete Wall

The detail shown in Figure 8-9 is applicable when it is desired to expose the exterior face of the concrete wall. The first floor wall is located inward so that 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.

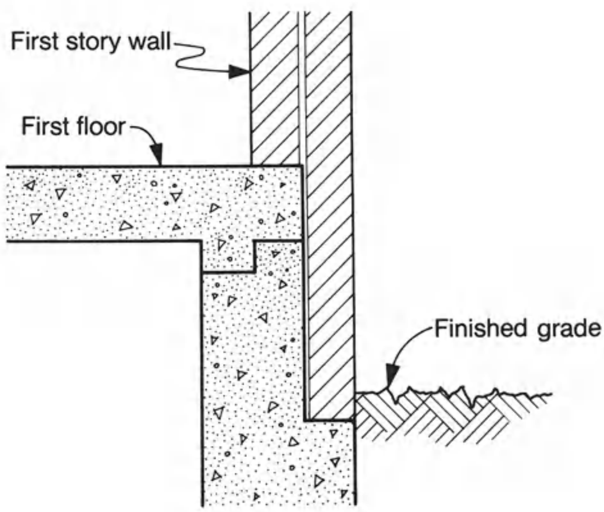


FIGURE 8-8. Basement wall with face brick below grade.

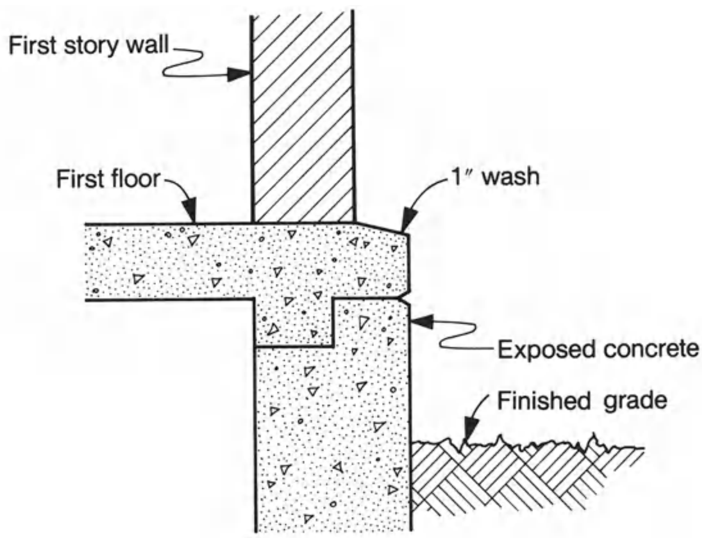


FIGURE 8-9. Basement wall with exposed concrete.

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.

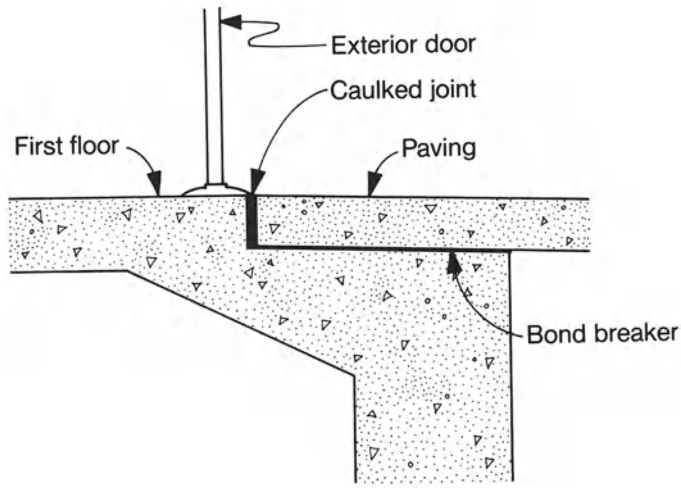


FIGURE 8-10. Basement wall with a recessed entrance.

Recessed Entrance

The detail shown in Figure 8-10 applies where the first floor wall has been moved inward from the face of the basement wall in order to create a recessed 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 a continuation of the first floor, interrupted only by the threshold. When the threshold is directly exposed to rain water then the paving should 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.

First Floor Slab Extension

This detail, as shown in Figure 8-11, is 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.

Basement Wall Bearing on a Continuous Footing

The detail shown in Figure 8-12 illustrates the construction where the basement wall bears on and receives vertical and horizontal support from a continuous wall

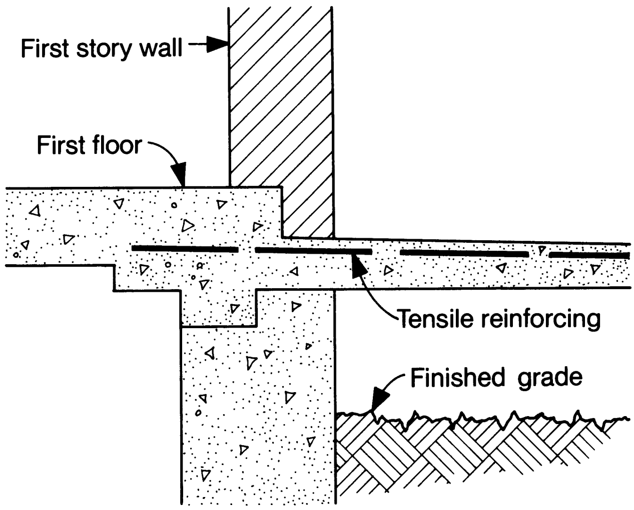


FIGURE 8-11. Basement wall with an extended first floor slab.

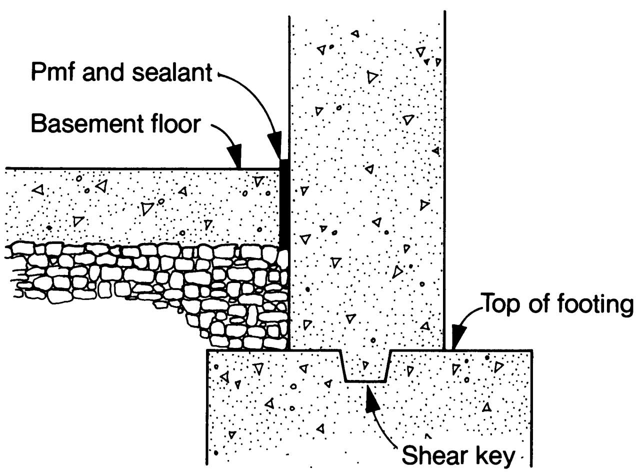


FIGURE 8-12. Earth pressure transfer from basement wall to footing.

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.

For details and allowable load transfers on this type of shear key refer to Appendix B.

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 4 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 detail shown in Figure 8-13 is representative of the construction which occurs when the basement wall does not bear on a continuous footing, but is designed to span as a beam between widely spaced footings. Such a wall is referred to as a *grade beam.* Earth pressure, in this instance, is usually 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 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

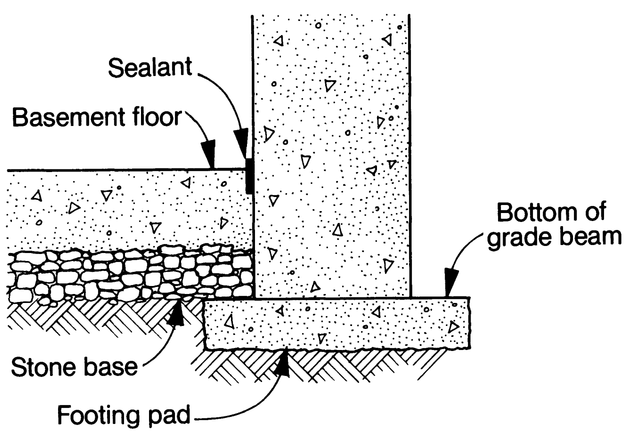


FIGURE 8-13. Earth pressure transfer from basement wall to slab on ground.

pressure as well as serving as a finished basement floor, the author believes that a little extra thickness should not be considered unreasonable.

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.

8-7. 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, and not for adding tensile strength. Concrete walls may be faced with stone, as required by architectural treatment of the area. A typical gravity wall is illustrated in Figure 8-14.

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 computed to provide adequate resistance against failure by overturning or by slide. In order to provide a reasonable starting point for the computations a good rule of thumb is to make the width of the base equal to one-half of the overall height of the

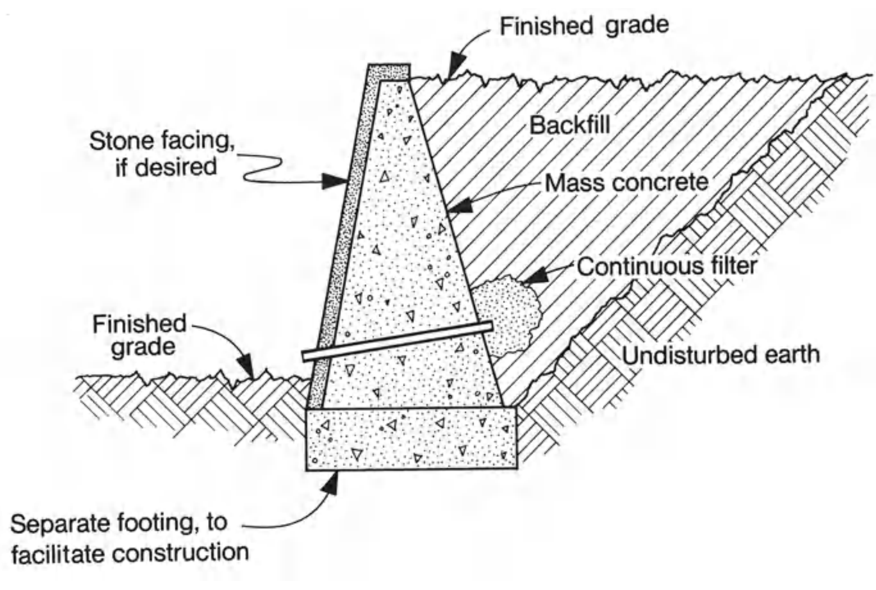


FIGURE 8-14. Typical detail of a gravity retaining wall.

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 insure 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 twelve 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.

8-8. 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 developments 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,

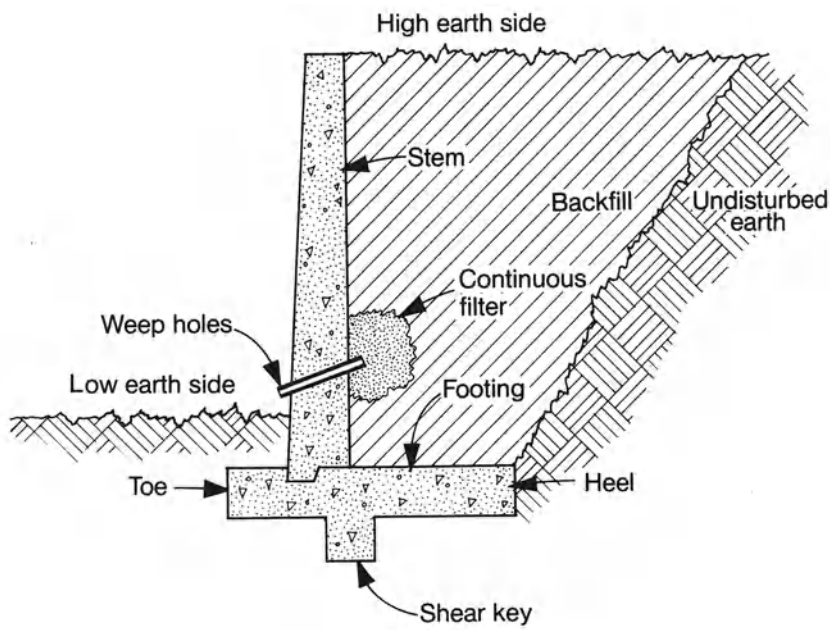


FIGURE 8-15. Typical detail of a cantilever retaining wall.

has designed walls having a height of approximately 50 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 8-15.

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 can be faced with brick or stone, 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 several factors:

1. The in-place character and density of the backfill
2. The surface of the restrained earth, as to whether it is level or sloped
3. The absence of, or the weight of, any surcharge
4. The positioning of the wall with respect to the footing

It must be noted that this is a sophisticated wall, requiring expertise in its design and construction.

8-9. WALLS REQUIRING SPECIAL RESTRAINTS

There are walls that simply cannot develop sufficient lateral resistance through a 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 these walls of special concern. There are three general procedures by which the amount of available resistance can be dramatically increased.

Battered Piles

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 be 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 of 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 along the length of the wall. Care must be taken, while determining the resistance to lateral load, 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.

The use of battered piles is illustrated in Figure 8-16.

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.

Prestressed Tiedowns

The second method whereby lateral load resistance can be increased is to anchor the footing down with prestressed tiedowns. This method is highly effective when the tiedowns can be anchored into bedrock or into soil that is extremely dense and hard. The installation of these tiedowns should follow the procedures outlined in Section 8-10. Tiedowns must be placed on the opposite side of the footing from that of the battered piles and must also be battered in the opposite direction. The reason for this, of course, is that these tiedowns are only effective in tension.

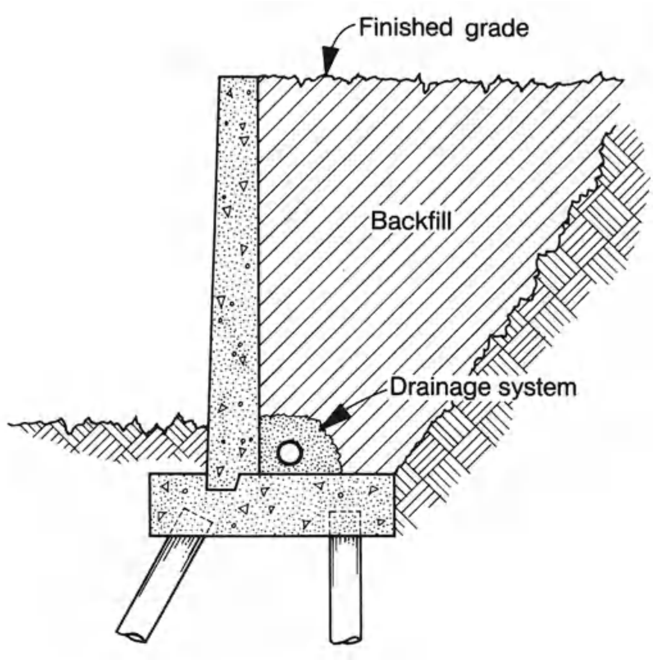


FIGURE 8-16. Retaining walls with battered piles.

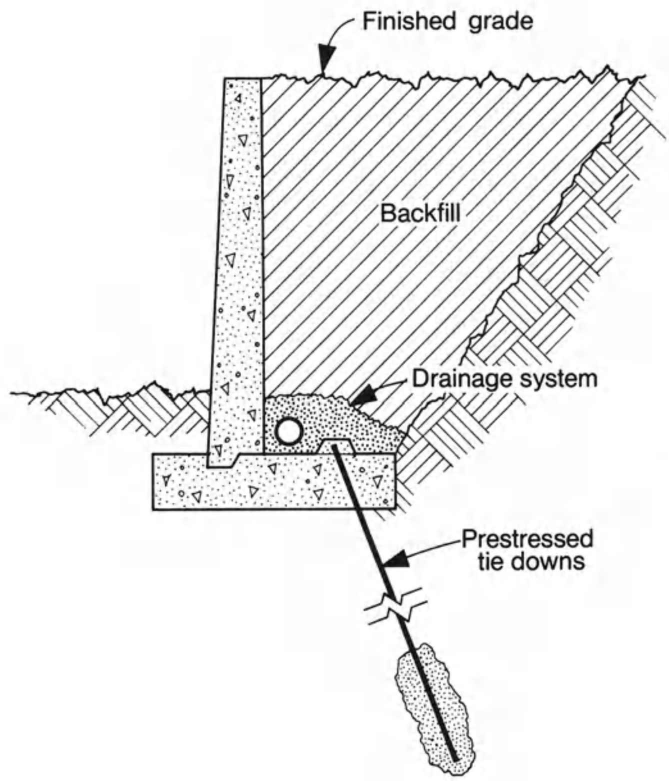


FIGURE 8-17. Retaining walls with prestressed tiedowns.

Tiedowns also give an added bonus in that they provide a substantial vertical component which acts to resist the effect of overturning. This type of arrangement is illustrated in Figure 8-17.

Prestressed Tiebacks

The third method of providing additional lateral earth resistance is to anchor the wall back into undisturbed earth with prestressed tiebacks, thereby removing all or most of the horizontal thrust from the footing. Tiebacks must be installed in accordance with the procedure outlined in Section 8-10. This method, of course, is feasible only when the earth will not collapse into the tieback holes before the ties have been installed and grouted. This is of special concern because this particular earth may lack sufficient stickiness or density. It must also be noted that this procedure is permissible only when the owner of the retaining wall has legal access to the adjacent underground property.

A typical arrangement of prestressed tiebacks is illustrated in Figure 8-18.

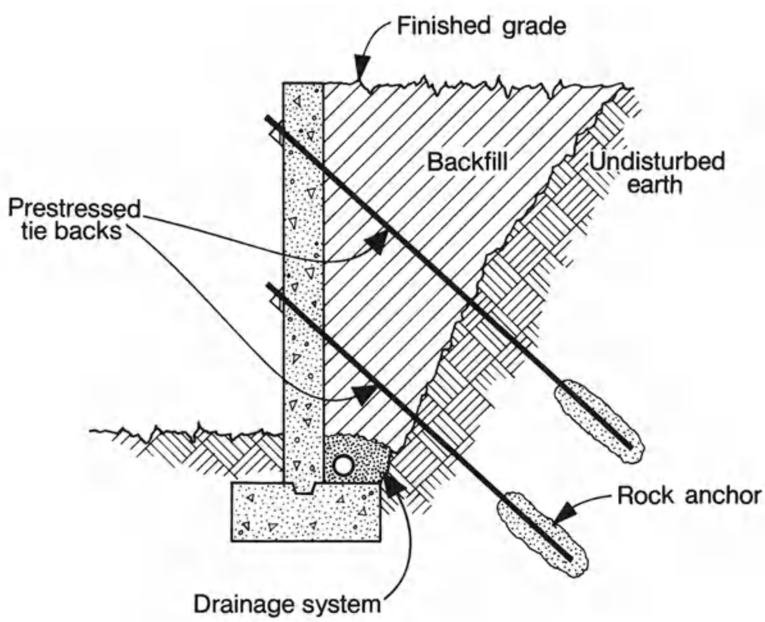


FIGURE 8-18. Retaining walls with prestressed tiebacks.

Although the tiedown system of Figure 8-17 and the tieback system of Figure 8-18 produce the same result, it is of interest to note the major differences between them. The tiedown system is truly a cantilever retaining wall, whereas the tieback system is that of a wall designed to span vertically between anchor points. The tiedown system, therefore, will require heavier reinforcing, a wider footing, and considerably more excavation. It should also be noted, however, that the cables in the tieback system are more difficult to install, and consequently more expensive.

8-10. PRESTRESSED TIEDOWNS AND TIEBACKS

Purpose

Prestressed tiedowns and tiebacks are high strength, post-tensioned cable assemblies which are used to apply an external force to a structure to provide additional restraint against lateral pressure or rotation. These cables are anchored into bedrock or extremely hard, dense soil with a grout pocket which transfers the force from the cable into the supporting rock or soil by friction. This end of the assembly is called the *dead end anchorage.* The cables are anchored to the structure by wedges and bearing plates so as to transfer their force directly into the structure. This end of the assembly is called the *live end anchorage.* Although not necessarily anchored into bedrock, these cables are frequently called *rock anchors.* Tiedowns and tiebacks are used for one or more 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 illustrated in Figures 8-3 and 8-5. These cables provide not only direct horizontal support, but also increase the vertical seating pressure at the base of the wall.
2. To provide permanent stability to a cantilever retaining wall by tying the footing down into the soil beneath, as indicated in Figure 8-17. This procedure increases the ability of the wall to resist rotation.
3. To provide permanent lateral support to a retaining wall by tying the wall back into the undisturbed soil at some distance behind the backfill, as illustrated in Figure 8-18.
4. To stabilize a footing subjected to lateral load, as illustrated in Example 9-9 of Chapter 9:
   a. By providing direct lateral support through the direct action of the horizontal component.
   b. To increase frictional resistance at the base of the footing by increasing the pressure of contact through the action of the vertical component.

It is evident that these cables are not always anchored into rock, but are sometimes anchored into very stiff, hard soil. The problems with anchorage into soil are twofold:

1. The soil must have sufficient strength to develop the required anchorage.
2. The soil must have sufficient consistency so that the side walls will stand intact until the entire operation is completed.

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, which has an ultimate strength in the range of 250 to 270 ksi. The effective prestress which can be developed by a single strand varies between 12.6 and 24.5 kips, depending upon the area of the strand and the ultimate strength of the wire.

Installation

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 drilling operation is shown in Figure 8-19.

It should be noted that in the background of this figure the reader can plainly see the soldier beams and timber lagging which were used during the construction of major excavation at this site.

In order to fully develop the cable, the cable must be adequately anchored into the ground at the dead end anchorage. This usually requires a grout pocket having a contact length of five to ten feet. Immediately prior to anchoring the cable the

FIGURE 8-19. Rock anchor hole being drilled. [Refs. 9 and 12]

hole should be cleaned out with compressed air or by flushing with water. The cable is then lowered into the hole to within about four inches of the bottom. A one inch diameter flexible grout tube is also lowered into the hole and a nonshrinking cement grout is pumped down into the area required for the grout pocket. This procedure just prior to grouting is shown in Figure 8-20.

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 which transfer the force from the cable into a bearing plate assembly which bears directly onto the concrete. Cable elongation is measured as shown in Figure 8-21.

FIGURE 8-20. Rock anchor cable and grout tube positioned in hole prior to grouting the dead end anchorage. [Refs. 9 and 12]

FIGURE 8-21. Measuring the rock anchor elongation after the cable has been prestressed. [Refs. 9 and 12]

Protection of Cables

All cables considered to be part of a permanent installation must be adequately protected against the adverse, long term action of soil, water, and air. Adequate protection may be obtained by encasing the cable for its full length with the same nonshrinking grout that was used for the grout pocket. This procedure requires that two grouting operations—one to set the grout pocket and one to provide encasement of the cable above the pocket. An alternative procedure is to provide a greased plastic sheath in which the cable can move freely. The sheath, however, cannot encroach into the grout pocket length, since this part of the cable must remain bare. With this arrangement the grout pocket and the grout encasement can be poured in one continuous operation.

Destruction Tests

The ultimate strength of the transfer of forces at the dead end anchorage can only be determined by testing a completed installation 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 then the design load can be confidently determined by applying the appropriate safety factor.

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.

The number of destruction tests required depends on the site and on the size of the project. Two tests are considered to be a reasonable minimum. Where test borings show consistency throughout the site fewer tests would be required than when there appears to be considerable variation in the rock or soil characteristics. This is another of the many reasons why test boring procedures are so very important to the overall performance of all those engaged in the design and construction of the project.

8-11. 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, then earth that is suitable must 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. The material must be available for delivery in the quantity required for the work to proceed without delay or interruption.
2. 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.
3. The material must be such that it can be compacted to the density required for its intended use.
4. The backfill must exhibit sufficient strength characteristics to support its own weight and the weight of any surcharge without undue settlement.
5. The material must have sufficient inherent permeability to insure adequate drainage of rain water behind the wall.
6. Soils with medium to high levels of shrink-swell potential, defined as soils having a plasticity index greater than 10, 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.
7. Backfill should be free of all extraneous materials such as roots, 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.
8. Backfill must be cleared of rocks, bricks, pieces of stone, bolts, nails, or any other hard, sharp material that could damage waterproofing or drainage systems or underground mechanical services.
9. 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.
10. 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 the later subsection on 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, SW, and SP of the Unified Soil Classification System meet all of the above characteristics and are considered to be superior materials for use as backfill. Refer to Sections 10-9 and Table 10-3 of Chapter 10, Soil Compaction.

Use of Cohesive Materials

Cohesive soils, particularly those consisting predominantly 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 10, should never be used as backfill since these clays can exert very large lateral pressures when restrained against free expansion. Refer to Chapter 11, Expansive Clay.
3. Clay deposits are relatively impermeable, which makes adequate drainage in back of the wall impossible. The wall, therefore, must 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.

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 eight 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.

For an in-depth discussion relative to the process of compaction refer to Chapter 10.

Design Responsibility

The successful long term performance of any retaining wall depends on the proper selection of material to be used as backfill and on the proper installation of that material. Two situations can arise in the selection of the backfill material:

1. When the architect and engineer have the prerogative of exercising control over the kind of material to be used, then it is their responsibility to make that selection during the design stage of the project and to further determine a source from which this material can readily be obtained.
2. When the selection of material is the prerogative of others, then it is the responsibility of the architect and engineer to require such tests as are necessary to satisfy them as to the suitability of the material for the purpose intended. If the material is found to be unsuitable, then it must be rejected.

In each of these instances, it is the further responsibility of the architect and engineer to determine the lateral pressure that will be imposed by the soil selected, and to design the wall for that pressure.

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 field inspection, and such soils testing as required by contract with the architect and engineer.

8-12. 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, then, must be designed to resist the resulting water pressure. When there is only incidental ground water behind the wall, then drainage 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 rain storm, 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

The drainage system most generally used around basement walls is illustrated in Figure 8-7, earlier in this chapter.

For this condition 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 can not 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 collectors:

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

The pipe must be completely surrounded by a thick envelope of selected sand and gravel called a *drainage filter,* the purpose and design of which is discussed in a later paragraph. Care must be taken during construction to insure 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 any drainage system be considered as negating the need to dampproof or waterproof the basement perimeter.

Drainage System — Cantilever Retaining Wall

The primary difference between the drainage of a cantilever retaining wall and that of a basement wall is the way in which the collected water is discharged from the continuous filter. The types of collectors used in building construction will not be available out in open areas. Discharge, therefore, must be accomplished by the use of either of two different methods.

The first method is applicable primarily when there is an open ditch in the immediate vicinity of the wall. The drainage system will be similar in all respects to the system used for basement walls except that the pipe drain will discharge its collected water, by gravity, into the open ditch. Once the water has been discharged it will flow away from the wall and dissipate into the ground. This is the preferred method, and should be used whenever possible.

The second method may be used when there is no ditch in which to discharge the collected water. This method utilizes 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 are placed no farther than eight feet apart and must be installed for 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 six inches in diameter and should be pitched two 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. A typical weep hole installation is shown in Figure 8-15.

The weep hole method of drainage is not as positive a system as the one in which the collected water is discharged into an open ditch. Due to the desirability of providing a positive method of drainage the principle of the open ditch should be used wherever possible. The following possibilities should be explored:

1. When the closest ditch is some distance away from the wall, it may be possible to extend the discharge pipe underground to the ditch.
2. When the surface of the ground in front of the wall slopes away from the wall, it may be possible to run a pipe underground and have it exit and discharge at some lower point on the surface.

Drainage System — Gravity Retaining Wall

The requirements of drainage for a gravity retaining wall are, in all respects, similar to those for the cantilever retaining wall. Drainage is better served by the first method, in which a continuous pipe is used for the collection of water. The second method, that of weep holes, is very popular with contractors due to its relative ease of construction and low cost. Note that when the open ditch method is used the footing shown in Figure 8-14 must be widened to accommodate the drainage system. This increase in footing width requires a corresponding increase in bulk excavation.

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 in 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, 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:

$$\frac{D_{15} \text{ of filter material}}{D_{15} \text{ of backfill}} > 4 \tag{8-1}$$

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 [Ref. 13]:

$$\frac{D_{15} \text{ of filter material}}{D_{85} \text{ of backfill}} < 5 \tag{8-2}$$

$$\frac{D_{50} \text{ of filter material}}{D_{50} \text{ of backfill}} < 25 \tag{8-3}$$

$$\frac{D_{15} \text{ of filter material}}{D_{15} \text{ of backfill}} < 20 \tag{8-4}$$

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 [Ref. 10]:

$$\frac{D_{85} \text{ of filter material}}{\text{Diameter of opening}} > 2 \tag{8-5}$$

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 a square. The particle, however, is not square, but 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.

8-13. SAMPLE PROBLEMS

Example 8-1

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 8-1.

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 of Figure 1-6 requires that the coefficients of uniformity and curvature be evaluated in order to classify the

TABLE 8-1. Percentage of Total Weight Passing.[a]

Sieve Size	3″	$\frac{3}{4}$	#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 produce the curve shown in Figure 8-22.

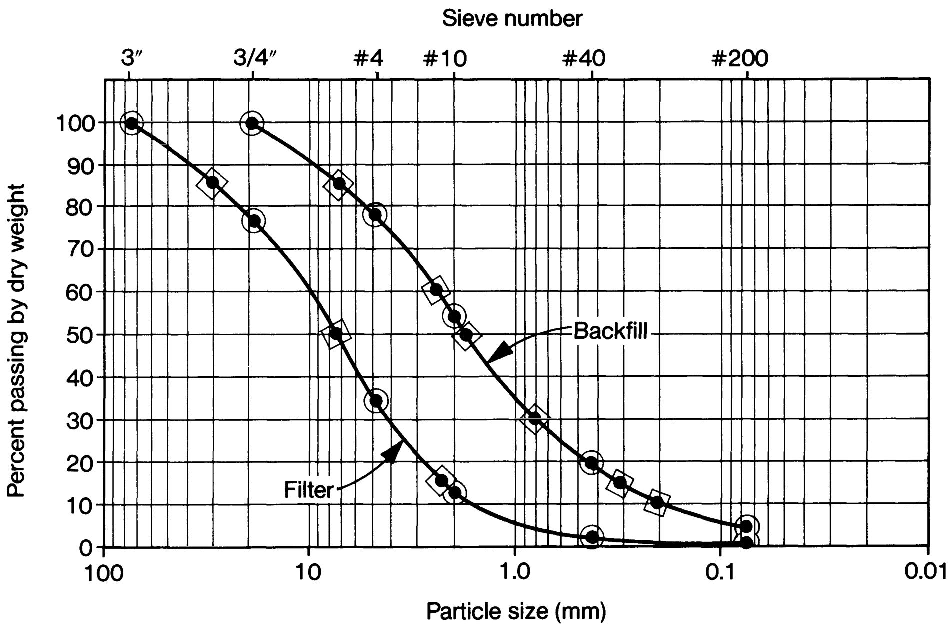


FIGURE 8-22. Points shown circled are plotted from Table 8-1, points shown boxed are read off of the curve and recorded in Table 8-2.

sample as well graded or poorly graded. The use of these coefficients is described in Section 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 on Figure 8-22.

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 8-2.

To determine the grading characteristics of the backfill, compute:

$$C_u = \frac{2.4}{0.20} = 12 \quad \text{and} \quad C_c = \frac{(0.80)^2}{2.4 \times 0.20} = 1.3$$

TABLE 8-2. 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 on Figure 8-22.

For well graded sand the coefficients must satisfy the following:

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

It can be seen that the backfill satisfies these requirements and can finally be classified as a well graded sand.

To determine the suitability 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:
From Formula (8-1):

$$\frac{2.3}{0.31} = 7.4 > 4 \quad \text{OK}$$

From Formula (8-2):

$$\frac{2.3}{7.2} = 0.32 < 5 \quad \text{OK}$$

From Formula (8-3):

$$\frac{7.4}{1.7} = 4.4 < 25 \quad \text{OK}$$

From Formula (8-4):

$$\frac{2.3}{0.31} = 7.4 < 20 \quad \text{OK}$$

The filter material, therefore, satisfies all requirements.

9

Walls—Design Considerations

9-1. 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.

Prior to construction of the wall, the immediate area must be excavated so as 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 in-situ characteristics of the backfill or of the undisturbed earth, depending on the following:

1. The width of excavation
2. The material through which the excavation is made

Lateral Pressure as a Function of Excavation

As described in Section 7-2, 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 noticeable 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 9-1 and further described herein.

Figure 9-1(a): 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 9-1(b): Excavation in cohesive soil. When excavating in stiff, cohesive soil, it may be determined that the side walls will remain intact 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 9-1(c). Excavation in non-cohesive 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, therefore, must remove sufficient earth so that the surface of the cut approximates the angle of repose of the soil. In 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, for one reason or another, to make an excavation having a relatively gentle slope.

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 9-1(b) would, therefore, normally

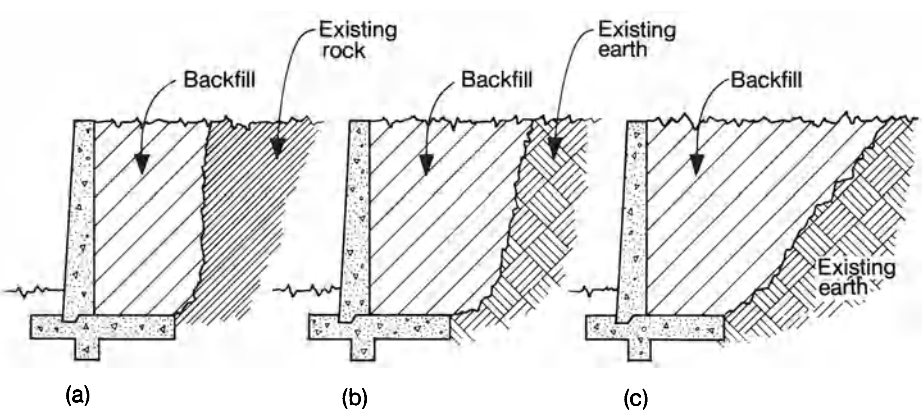


FIGURE 9-1. Source of lateral pressure on a retaining wall.

be preferred by the contractor, and he may choose to use it as long as the side wall remains stable. If this wall starts to spaul off or to cave in, the contractor has two options:

1. Remove more earth, thereby reducing the slope of the excavation.
2. Shore the excavation until the work is complete.

The ability of the earth to stand without caving in can be approximated from information obtained from the borings. The sides of stiff, cohesive soils 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 which dry out are especially susceptible to failure. This can be of serious concern, particular in hot weather, because of the extent of the exposure of the side walls to atmospheric conditions.

When test borings indicate that the future excavation will be in earth, there will be uncertainty, at the time of design, as to what width of excavation will ultimately result. Prudent engineering calls for a safe approach in determining the lateral earth pressure for which the wall should be designed. It is recommended, therefore, that the lateral pressure characteristics of both the backfill and the existing earth be determined, and that the larger of these be used for design.

For an in-depth discussion regarding the determination of the lateral pressure characteristics for different soils, refer to Chapter 7.

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 at and below this point is known as *ground water.* This is the water that one will find when digging a well. Beneath this level all of the voids and pores within the soil are completely filled with water. Due to a phenomenon known as *capillary attraction,* 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 buildup 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 buildup 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, then the wall must 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 is lowered (drawn down) in the immediate vicinity of the wall. If the water table is actually lowered then the wall need not be designed to resist water pressure. The problem here is that the table must be lowered permanently and no buildup of water pressure can be permitted, even on a temporary basis. It must be noted that water will constantly seep into the backfill from the ground water present in the adjacent soil. It seems to the author that this is asking a great deal of a drainage system.

9-2. 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 supports 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.

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 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.

When a wall is designed to span vertically it will act as a single span element. The lateral load for which this wall must be designed is a function of the conditions existing outside of the wall, including the type of backfill, the existence of ground

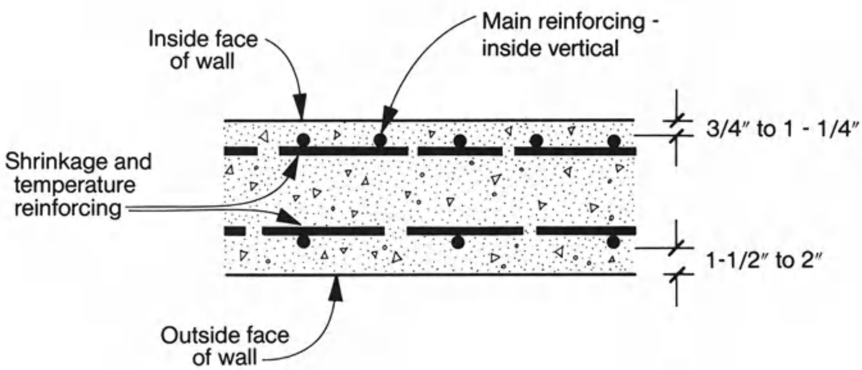


FIGURE 9-2. Typical wall reinforcing, for vertical design.

water, and the presence of surcharge. A number of loading diagrams, covering all reasonable conditions, have been given in Section 7-8.

The reinforcing that would normally be expected for a basement wall designed to span vertically is shown in Figure 9-2.

The inside vertical bars are designed to resist the tensile stresses induced by the positive bending moment. They should extend for the full height of the wall without being spliced. The other three layers of bars are shrinkage and temperature reinforcement.

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 of 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.

The reinforcing that would normally be expected for a basement wall designed to span horizontally is shown in Figure 9-3.

When walls are designed to span horizontally, they are in fact, continuous spans. There will be a positive moment at the center of each span, and a negative moment at each support. Both layers of horizontal bars, therefore, will carry tensile stress. These bars, of course, can not be fabricated in one continuous piece for the full length of the building, but the splices must be located at the points where the bending moment causes tension on the opposite side of the wall. The two layers of vertical bars are shrinkage and temperature reinforcing.

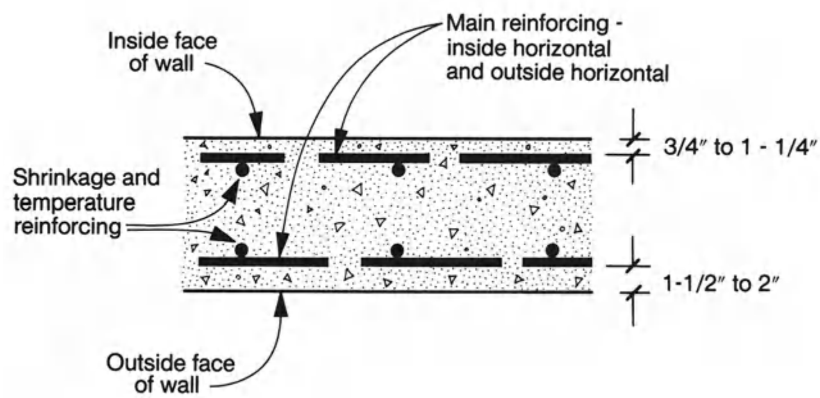


FIGURE 9-3. Typical wall reinforcing, for horizontal design.

General Details of Reinforcement

As indicated on the preceding details, reinforcement is customarily placed horizontally and vertically in both faces of the wall. Some of this reinforcement is for the purpose of providing tensile strength to the wall in the direction of bending. The remaining reinforcement is for the purpose of controlling the incidence of cracking as caused by drying shrinkage or change in temperature. It should reasonably be expected that the former reinforcement will be larger or more closely spaced than the latter. Minimum reinforcing, in accordance with the ACI 318-83 Building Code, is given in Table 9-1.

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.

TABLE 9-1. Minimum Reinforcing for Basement and Retaining Walls.

Wall Thickness	Horizontal in Each Layer	Vertical in Each Layer
8″	#3 @ 12 or #4 @ 18	#3 @ 16
10″	#4 @ 18	#3 @ 16
12″	#4 @ 16	#3 @ 12 or #4 @ 18
14″	#4 @ 12 or #5 @ 18	#3 @ 12 or #4 @ 18
16″	#5 @ 18	#4 @ 18

Note: The size and spacing of reinforcing as given in this table is valid only for ASTM-A615, Grade 60 reinforcement.

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. Assume, for sake of discussion, that the earth pressure is without surcharge or ground water:

1. The vertically designed wall, whose strips are divided horizontally, will carry a triangular load. The intensity of this load will remain constant for each of the strips.
2. The horizontally designed wall, whose strips are divided vertically, will carry a trapezoidal load. The intensity of this load will vary from strip to strip, becoming less and less in upper strips. It would be impractical to design each strip individually. Therefore, there must be a certain amount of grouping, with the resultant overdesign in 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 9-1 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 are 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, then 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 which preclude the use of the slab on ground follow:

1. The floor slab may be subjected to climatic changes in temperature, as would be the case in an open air garage. The floor, then, must 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 underground utilities.
4. The sequence of construction necessitates that the wall be 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, then it must 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. 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 be impractical, then, to support the basement wall on a continuous footing. The wall must be designed to satisfy two different conditions:

1. To transfer all gravity loads to building elements which extend down to satisfactory bearing. The wall, in this instance, will be designed as a deep beam, commonly referred to as a *grade beam.*
2. To transfer the earth pressure loads into the slab on ground (vertical design) or into the building columns (horizontal design).

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 is illustrated in Figure 8-13 of Chapter 8.

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.

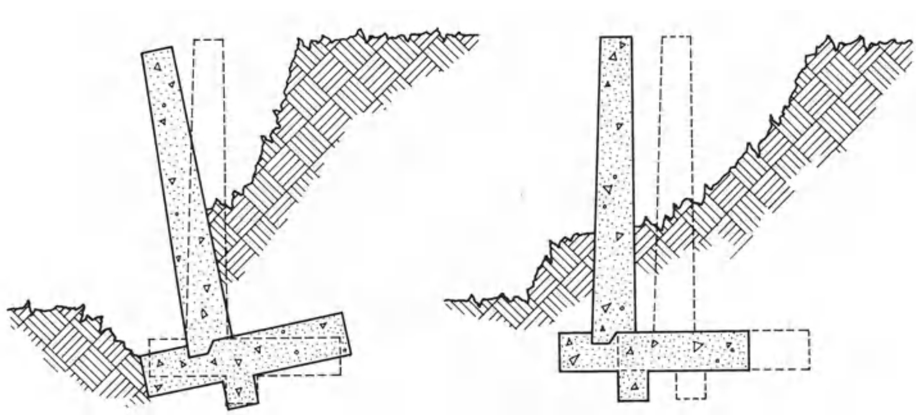
9-3. CANTILEVER RETAINING WALL— MODES OF FAILURE

The response to lateral earth pressure by a cantilever retaining wall is completely different from that of a basement wall. The basement wall, it will be remembered, has two supports regardless of whether it is designed vertically or horizontally. The cantilever retaining wall only has one support—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 9-4.

Overturning Mode

A wall will fail by overturning when the overturning effect of the lateral earth pressure exceeds the capacity of the wall to resist. This resistance is a function of the following:



(a) (b)

FIGURE 9-4. Failure modes of a cantilever retaining wall, where (a) indicates failure due to overturning, and (b) indicates failure due to sliding.

1. The weight of the wall, including stem and 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 would not fully develop until a certain amount of deformation would occur within the soil mass behind the wall. Because this deformation would allow for some degree of wall rotation, this resistance is 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 to resist. This resistance, as developed solely by the footing, is a function of the following:

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.

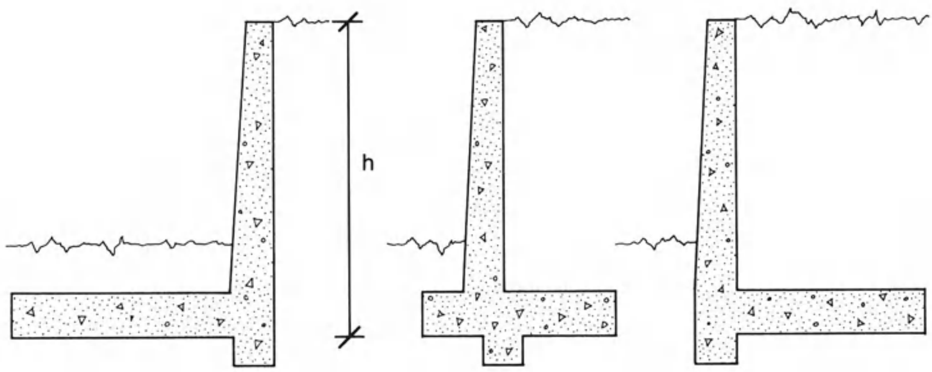
In heavily loaded walls a shear key may be required to add to the resistance of the against sliding. The key should preferably be located near the toe of the footing because of the greater intensity of vertical pressure at that point. A key may also be used for anchorage of the main tensile reinforcing from the stem. Such a key is illustrated in Figure 9-11 in a subsequent paragraph.

9-4. 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 9-5. 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 back-fill and any change in loading parameters. They should be used, therefore, only as a guide as to where the calculations might begin.

Type 1

Walls in this category are the least efficient of all three types of wall in terms of engineering design. The earth which bears on the heel of the footing 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.



Type 1 Type 2 Type 3

FIGURE 9-5. Different orientation of cantilever retaining walls, in which the width of the footing may be taken as $0.70h$ for Type 1, $0.50h$ for Type 2, and $0.60h$ for Type 3.

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 of wall. 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 at least 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 previously given, but then rounded off, preferably 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.
4. The thickness of the footing can be approximated as one-eighth 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.

9-5. TYPICAL REINFORCING—CANTILEVER RETAINING WALL

The typical reinforcing of a cantilever retaining wall is shown in Figure 9-6. Note the main tensile reinforcing in the stem is placed vertically along the inside face. This is because of the bending moment produced by the cantilever action of the

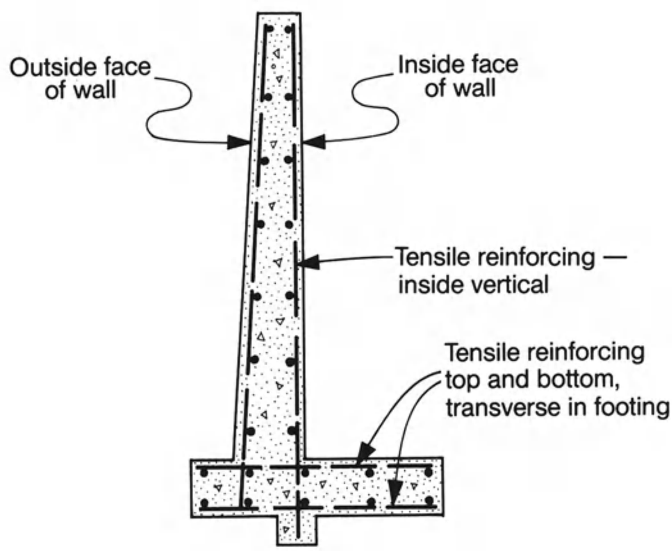


FIGURE 9-6. Typical reinforcing for a cantilever retaining wall.

stem. 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 the tensile reinforcing of the stem has been extended into the shear key beneath the footing. The reason for this is to provide sufficient length to fully develop the bars. When there is no key, or when it is not positioned to receive the bars, then that reinforcing must be hooked into the toe of the footing for development.

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 9-1.

9-6. COUNTERFORT RETAINING WALLS

In walls up to about twenty feet in height the vertical element of the wall, called the *stem,* is designed as a vertical cantilever off of 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 9-7.

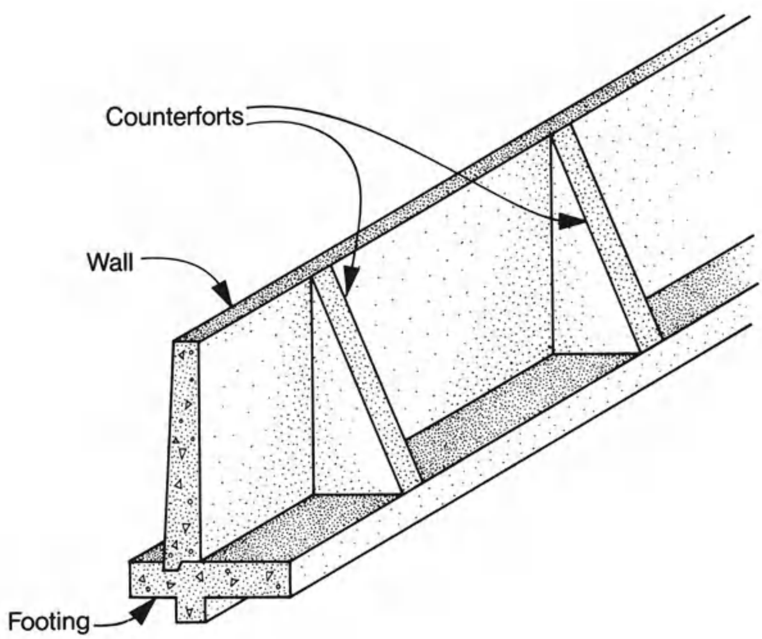


FIGURE 9-7. Counterfort retaining wall.

9-7. 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, through a variety of different 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, all acting 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:

Type 1. Between basement wall and first floor slab
Type 2. Between basement wall and footing, and between footing and stem of cantilever retaining wall
Type 3. Between basement wall and basement slab

Types 1 and 2

Transfer of earth pressures across these types of joints can be analyzed by using either of the following procedures:

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. For a detailed analysis of this method refer to Appendix A.
2. *Shear key design,* in which transfer is made by a combination of three separate resisting forces: (a) pure shear (as opposed to diagonal tension) acting on the horizontal shear plane, (b) flexure acting on the same horizontal plane, and (c) bearing acting on the vertical surface of contact. For a detailed analysis of this method refer to 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.

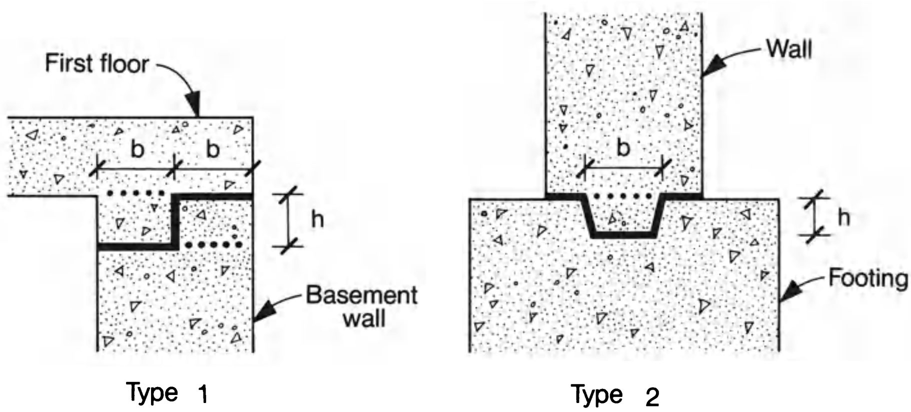


FIGURE 9-8. Cold joint details—types 1 and 2.

The shear key method of earth pressure transfer across joint Types 1 and 2 is illustrated in Figure 9-8. 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.

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 the sides of the shear key are sloped. This is to permit easy removal of the formboard after the concrete has set. This slope need not occur on both sides, although contractors usually prefer to slope both sides for simplicity. Where the key must transfer heavy pressures the designer may wish to specify a key having only one side sloped, as in Figures 8-7 and 8-15 of Chapter 8.

Table 9-2 gives safe values for the force which can be transferred across a concrete shear key having width *b* and height *h*, as identified in Figure 9-8. This

TABLE 9-2. Allowable Transfer Force on Concrete Shear Keys.

Width *b*	Height *h*	Transfer Force *P*[a]
4″	2″	2160 #
6″	3″	3600 #
8″	4″	5040 #
10″	5″	6480 #
12″	6″	7920 #

[a] Transfer force has been computed in terms of pounds per linear foot of key, with shear design governing, as per formula (B-4).

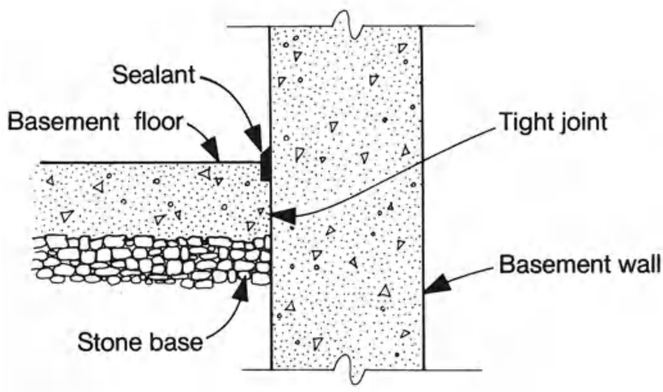


FIGURE 9-9. Cold joint detail—type 3.

table (which is a repeat of Table B-3) is applicable for concrete strengths of 3000 psi or greater.

Type 3

This type of joint, as shown in Figure 9-9, 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 against the wall so as to provide a surface of contact through which the load can be transferred. It is not necessary to check the resultant bearing stress on this surface because of the relatively high stress permitted for concrete in bearing:

$$f_{brg} = 0.3 f'_c \qquad \text{(per ACI Code, paragraph B.3)}$$

As an example, consider a 5 inch thick slab, with 2,500 psi concrete:

$$\text{Transfer force} = 0.3 \times 2{,}500 \times 5 \times 12 = 45{,}000 \text{ \#/linear foot}$$

Obviously, this is a much higher capacity than could reasonably be required by any load transference.

9-8. 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 transfer is made through a combination of two separate interactions between the earth and the footing.

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 due to passive earth pressure developed between the vertical side of the footing and the earth against which it bears.

These combined effects are illustrated in Figure 9-10,

Where:
W is the total weight supported by the footing,
P_a is the proportional share of the earth pressure on the wall which must be transferred into the earth by the footing,
F is the total frictional resistance developed between the footing and the earth, as given by Formula (9-2),
p_p is the passive pressure developed by the earth, as given by Formula (9-3), and
P_p is the total passive force developed by the earth, as given by Formula (9-4).

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.

It should be noted that the cold joint shown in Figure 9-10 was constructed differently than the one shown in Figure 9-8. Different details are always possible, and each must be tailored to suit a particular situation.

Adequate transfer of the lateral forces is absolutely essential to the performance of any earth retaining structure. It is particularly important that lateral movement of the footing be prevented. Excessive movement will cause failure. Lesser move-

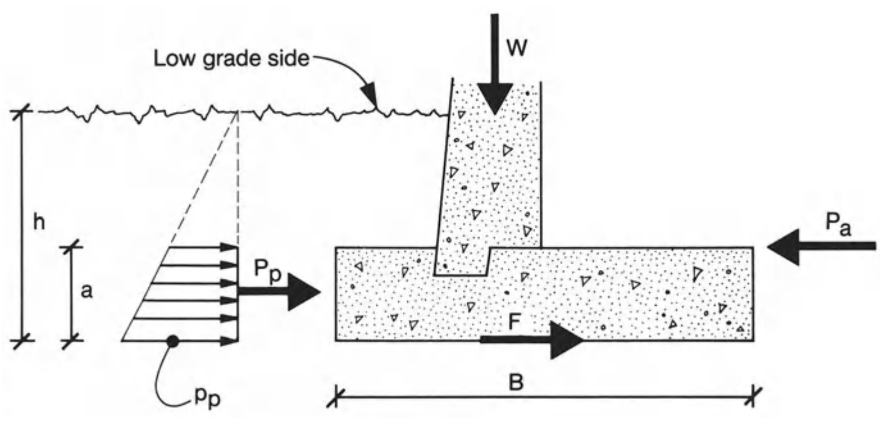


FIGURE 9-10. Earth pressure transfer, from footing to ground.

ment 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, as follows:

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 an internal resistance to shear by 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 which holds the surfaces in contact.

Cohesive soils develop an internal resistance to shear through a chemical bond 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 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 the equation generally referred to as *Coulomb's equation.* An in-depth discussion relative to this equation may be found in Section 7-3 of Chapter 7.

Coulomb's equation for resistance to shear, as developed in Article 7-3, is as follows:

$$s = c + p \tan \phi \tag{9-1}$$

Where:

s is the intensity of resistance to shear,
c is the unit cohesion,
p is the intensity of pressure normal to the surface, and
ϕ is the angle of internal friction.

Note: It is usually on the conservative side to take the cohesion as numerically equal to one-half of the unconfined compression strength, q_u.

Formula (9-1) 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 9-10:

$$F = cB + W \tan \phi \tag{9-2}$$

Where:

F is the total frictional resisting force,
cB is that part of the total force attributable to the cohesive fraction of the soil,
W is the total weight normal to the surface of contact, and
$W \tan \phi$ is that part of the total force attributable to the granular fraction of the soil.

Note: Stresses are computed on the basis of pounds per square foot of area, but forces are computed on the basis of the number of pounds acting on one linear foot of wall.

The general equation for determination of frictional resistance is:

$$\text{coefficient of friction} = \tan \phi = \frac{\text{frictional force } F}{\text{normal force } W}$$

When determining the frictional resistance of a granular soil, it should be noted that the coefficient of friction is numerically equal to the tangent of the angle of internal friction. For preliminary design prior to soil analysis the coefficients given in Table 9-3 may be used as guidelines.

In those instances when the soil beneath the footing consists predominantly of clay, so that the soil exhibits a measurable cohesion, then the surface should be roughened before the footing concrete is poured. When the soil is particularly stiff or hard the surface should be heavily indented with a pick and shovel rather than just being roughened. The purpose of this requirement is to provide a physical, frictional interlocking between the footing and the soil.

TABLE 9-3. Guidelines for Coefficients of Friction for Various Soils.

Soil Description	$\tan \phi$
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.

Resistance Developed by Passive Pressure

Resistance to lateral movement will be developed by passive pressure, as shown in the foregoing detail on earth pressure transfer between footing and ground. The magnitude of this resistance may be computed as follows:

$$p_p = K_p \gamma h, \text{ psf} \quad (9\text{-}3)$$

(The numerical value of this pressure is not required when computing the total force, but is given here for reference.)

$$P_p = K_p \gamma a (h - a/2), \text{ \#/ft} \quad (9\text{-}4)$$

Where:

K_p = the coefficient of passive pressure,
= $\tan^2 (45° + \phi/2) = 1/K_a$
K_a = the coefficient of active pressure,
described in Section 7-6 of Chapter 7. (9-5)

Note: The angle of internal friction used in passive pressure calculations relates specifically to the earth located to the side of the footing and not to the earth comprising either the backfill or the earth directly beneath the footing.

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 prior to excavation. Mixed grained soils will stand or fall depending on the proportions of the constituents.

The contractor may elect to 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 Section 5-2 of Chapter 5. The purpose of these screeds is to precisely outline the excavation and to prevent spalling 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 any spalled earth is filled and compacted in accordance with the requirements given in Sections 10-3 and 10-5 of Chapter 10.

Additional passive pressure can be developed by the use of shear keys, as shown in Figure 9-11. 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 always 2 or more.

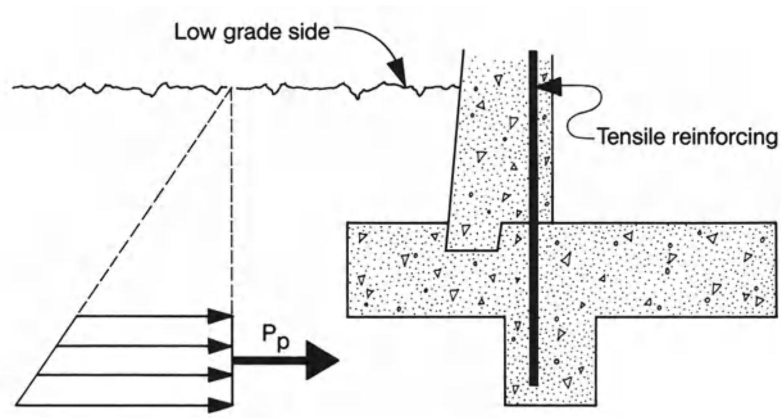


FIGURE 9-11. The development of passive pressure by the use of shear keys.

Shear keys are very effective when cut into rock. Their effectiveness in soils depends on whether the side walls of the excavation will remain during the construction process. The side walls of shear keys in soils must be carefully dug and then protected against cave-in as in the case with side walls of footings.

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 9-10, and the additional passive pressure developed by a shear key, if one is installed beneath the footing, as shown in Figure 9-11.

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 things which have a degree of uncertainty. It is for this reason that the writer recommends the use of a safety factor in the order of 1.8 to 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.

9-9. EARTH PRESSURE TRANSFER—BASEMENT SLAB TO GROUND

When the basement wall transfers earth pressure into the slab on ground, as indicated in the Type 3 cold joint illustrated in Figure 9-9, the pressure must first be transferred into the stone base, and from that base into the ground.

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 quasi-monolithic nature of the base. Because of this dual interaction, the slab and the stone base will act essentially as a unit to transfer the earth pressure to the ground.

Transfer of lateral pressure from the stone base into the earth beneath 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, as was done in Section 9-8, when the shear resistance between footing and ground was computed. The terms of that equation, as copied below, are redefined to express the total shearing resistance developed on a one square foot of contact area:

$$s = c + p \tan \phi$$

Where:

s is the total resistance to shear, psf,
c is the resistance produced by the cohesive fraction of the soil, psf, and
$p \tan \phi$ is the resistance produced by the granular fraction of the soil, psf.

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 of about 2 tsf. By applying a safety factor of 3, and remembering that cohesion is numerically equal to approximately one-half of the unconfined compression strength, the shearing resistance of this soil, after roughening, can be computed as:

$$s = c = \frac{0.5 \times 2 \times 2000}{3} = 670 \text{ psf}$$

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. The amount of friction that can be developed on this lower plane is a function of two things:

1. The angle of internal friction of the soil, of which approximate values have been given in Table 2-3 and Figure 2-4 of Chapter 2.
2. The contact pressure acting normal to the surface of shear. In this particular instance this pressure is equal to the combined weights of the basement slab and the stone base.

Transfer of earth pressure by friction alone will usually prove to be inadequate because the slab and stone base generally contribute insufficient weight to develop any appreciable frictional resistance. For example, a 5 inch slab with a 4 inch stone base will, at most, weigh 100 psf. Even with a coefficient of friction of 0.55, the resulting force of friction, from Formula (9-2), is only:

$$F = W \tan \phi = 100 \times 0.55 = 55 \text{ psf}$$

As a means of evaluating this frictional force it can be determined that a 12 foot high basement wall, supporting level earth without surcharge and without water pressure, may develop an earth pressure of about 2400 #/ft of wall, of which 1600 #/ft must be transferred at the base. The length of slab in order to make the transfer would be:

$$L = \frac{1600}{55} \times 1.5 \text{ (minimum safety factor)} = 44 \text{ feet}$$

In a normal building situation this length would be impractical.

This situation is indicative of a potentially serious problem which 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 through the slab. A completely different design concept for the transfer of earth pressure must then be developed. It should be evident that situations like this should be known well in advance of design commitment. This is one of the functions of subsoil investigation and shows one of the reasons why such an investigation should be scheduled well in advance of final design.

9-10. SAMPLE PROBLEMS

Example 9-1

Required: To determine the shear keys required to transfer the indicated lateral loads from the basement wall into the building structure, assuming vertical wall design. See to Figure 9-12. The backfill is a medium dense sandy gravel, having the following properties:

$$\gamma = 120 \text{ pcf} \quad \text{and} \quad \phi = 32^\circ$$

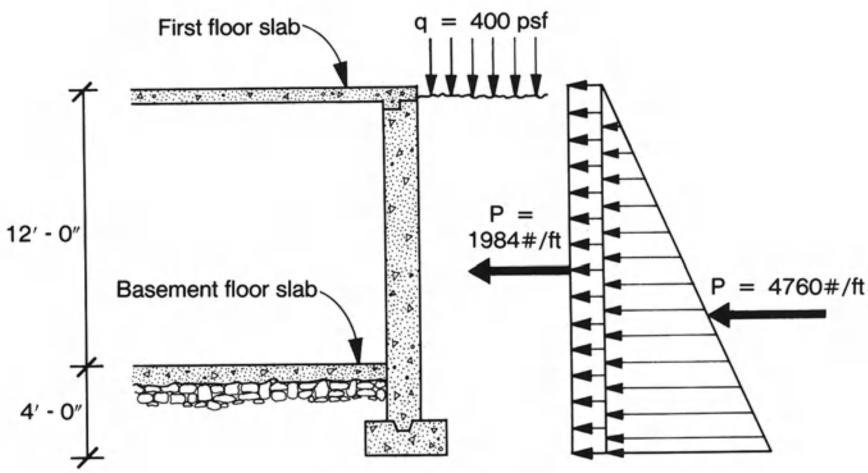


FIGURE 9-12.

From Formula (7-8):

$$K_a = \tan^2(45° - 32°/2) = 0.31$$

With reference to Figure 7-7:

(a) $K_a q = 0.31 \times 400 = 124$ #/ft
(b) $K_a \gamma h = 0.31 \times 120 \times 16 = 595$ #/ft
Total force on wall $= 124 \times 16 + 0.5 \times 595 \times 16$
$= 1984 + 4760 = 6744$ #/ft

Shear key transfer forces:
Top of wall:

$$P = \tfrac{1}{2} \times 1984 + \tfrac{1}{3} \times 4760 = 2579 \text{ \#/ft}$$

Bottom of wall:

$$P = \tfrac{1}{2} \times 1984 + \tfrac{2}{3} \times 4760 = 4165 \text{ \#/ft}$$

With reference to Figure 9-8 and Table 9-2:
Shear key at top of wall:

6″ wide × 3″ high

Shear key at bottom of wall:

8″ wide × 4″ high

Example 9-2

Required: To determine the adequacy of the footing in Example 9-1 to transfer the required lateral load. See Figure 9-13. The subgrade upon which the footing bears is a dense sandy gravel, having the following properties:

$$\gamma = 125 \text{ pcf} \quad \text{and} \quad \phi = 34°$$

Note: Previous computations indicate that the vertical load which the footing must carry is 5000 #/ft. This load includes the weight of the first story wall, the first floor slab, the basement wall and the footing itself.

Referring to Figure 9-10, resistance developed by shear, computed from Formula (9-2):

$$F = 0 + 5000 \tan 34° = 3372 \text{ #/ft}$$

Note the absence of the term involving cohesion.

Resistance developed by passive pressure, using Formula (9-4), with the coefficient of passive pressure K_p computed from Formula (9-5):

$$K_p = \tan^2 (45° + 34°/2) = 3.54$$
$$P_p = 3.54 \times 125 \times 1.5\ (4.00 - 0.75) = 2157 \text{ #/ft}$$

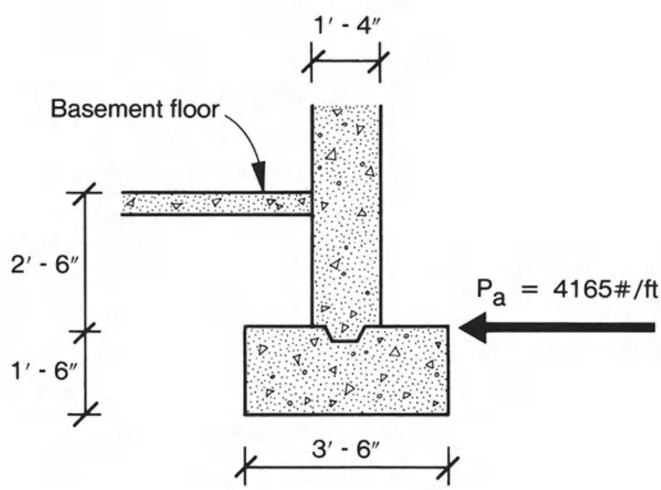


FIGURE 9-13.

The total resistance to slide, therefore, is 5529 #/ft, and the resultant safety factor is:

$$SF = \frac{5529}{4165} = 1.32$$

This design, therefore, is unsatisfactory.

Example 9-3

Required: To reexamine Example 9-2, assuming the soil in which the footing is to be poured is a silty clay mixed with sand. The properties of this soil are:

$$\gamma = 125 \text{ pcf}, \qquad \phi = 18°, \quad \text{and} \quad q_u = 1.20 \text{ tsf}$$

Referring to Figure 9-10, resistance developed by shear, computed from Formula (9-2):

$$F = 0.5 \times 1.20 \times 2000 \times 3.5 + 5000 \tan 18° = 5824 \text{ #/ft}$$

Note the use of both terms in this computation.

Resistance developed by passive pressure, using Formula (9-4), with the coefficient of passive pressure computed from Formula (9-5):

$$K_p = \tan^2 (45° + 18°/2) = 1.89$$

$$P_p = 1.89 \times 125 \times 1.5\,(4.00 - 0.75) = 1152 \text{ #/ft}$$

The total resistance to slide, therefore, is 6976 #/ft, and the resultant safety factor is:

$$SF = \frac{6976}{4165} = 1.67$$

This design is borderline satisfactory.

Some codes and some soils engineers accept a 1.50 safety factor against slide as being an acceptable minimum, in which case the above design is satisfactory. It is the opinion of the author, however, that this safety factor is too low, and should be increased to a minimum value of 1.8, but with a recommended value of 2.0.

Example 9-4

Required: Redesign Example 9-2, using a cohesive subgrade as a means of comparison. Assume the subgrade to be a medium stiff clay, having the following properties:

$$\gamma = 125 \text{ pcf}, \qquad \phi = 0°, \quad \text{and} \quad q_u = 2.20 \text{ tsf}$$

Referring to Figure 9-10, resistance developed by shear, computed from Formula 9-2:

$$F = 0.5 \times 2.20 \times 2000 \times 3.5 + 0 = 7700 \text{ \#/ft}$$

Note the absence of the term involving friction.

Resistance developed by passive pressure, using Formula (9-4), with the coefficient of passive pressure computed from Formula (9-5):

$$K_p = \tan^2 (45° + 0°/2) = 1.00$$

$$P_p = 1.00 \times 125 \times 1.5 \, (4.00 - 0.75) = 609 \text{ \#/ft}$$

The total resistance to slide, therefore, is 8309 #/ft, and the resultant safety factor is:

$$SF = \frac{8309}{4165} = 1.99$$

This design would usually be considered satisfactory.

Example 9-5

Required: To determine the stability against overturning for the cantilever retaining wall shown in Figure 9-14. The lateral earth pressure for which this wall is to be analyzed were previously computed in Example 7-2 of Chapter 2.

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 is computed as follows:

$$OM = 1736 \times \tfrac{1}{2} \times 14 + 3339 \times \tfrac{1}{3} \times 14 = 27734 \text{ ft\#/ft}$$

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 9-4.

The summation of moments about point a is as follows:

$$M_a = 69398 - 27734 - 13625 \, (x) = 0$$

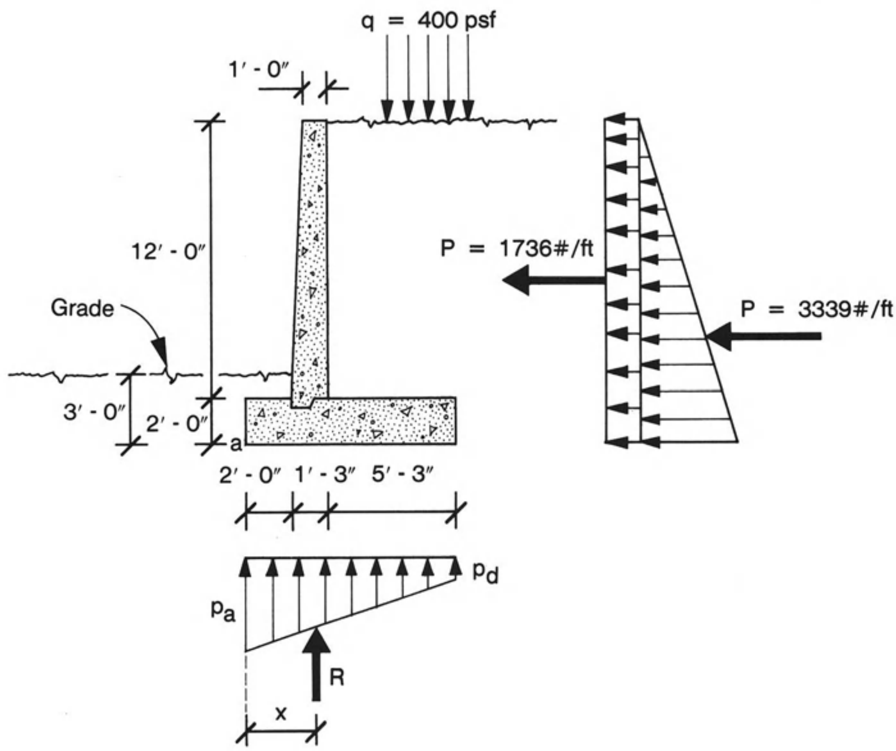


FIGURE 9-14.

From which:

$$x = 3.06 \text{ ft}$$

Applying the middle third rule:

$$\tfrac{1}{3} \times (8.5) < x < \tfrac{2}{3} \times (8.5)$$

TABLE 9-4. Computation of Gravity Loads and Righting Moment.

Item	Area	Weight[a]	Arm	Moment[b]	Source
1	5.3	2120	5.87	12444	surcharge
2	63.0	6930	5.87	40679	backfill
3	12.0	1800	2.75	4950	wall
4	1.5	225	2.17	488	wall
5	17.0	2550	4.25	10837	footing
		13625 #		69398 ft-#	

[a] Weights have been computed in pounds per linear foot of wall.
[b] Moments have been computed in foot-pounds per linear foot of wall.

Since x falls within the middle third of the footing width, the wall is stable against overturning.

Example 9-6

Required: To continue Example 9-5 to determine the soil pressure gradient induced into the soil beneath the footing.

The general formula with which to compute the stresses induced by the combined action of axial and bending stress is:

$$f = \frac{P}{A} \pm \frac{Mc}{I}$$

and for this example:

$$p = \frac{13605}{8.5} \pm \frac{13605\,(4.25 - 3.06)}{8.5^2/6} = 1600 \pm 1344$$

Therefore:

$$p_a = 1600 + 1344 = 2944 \text{ psf}$$

and:

$$p_d = 1600 - 1344 = 256 \text{ psf}$$

Example 9-7

Required: To compute the resistance against slide for the wall of Example 9-5. The subgrade into which the footing will be cast is a nonplastic silty sand, having the following properties:

$$\gamma = 115 \text{ pcf} \quad \text{and} \quad \phi = 28°$$

Referring to Figure 9-10, resistance developed by shear, computed from Formula (9-2):

$$F = 0 + 13625 \times \tan 28° = 7244 \text{ \#/ft}$$

Resistance developed by passive pressure, using Formula (9-4), with the coefficient of passive pressure obtained from Formula (9-5):

$$K_p = \tan^2 (45° + 28°/2) = 2.77$$

$$P_p = 2.77 \times 115 \times 2.0\,(3.0 - 2.0/2) = 1274 \text{ \#/ft}$$

The total resistance to slide, therefore, is 8518 #/ft, and the resultant safety factor is:

$$SF = \frac{8518}{5075} = 1.68$$

This is a borderline safety factor, and a redesign should be seriously considered.

Example 9-8

Required: To determine the effect that a shear key would have on the sliding resistance of the wall of Example 9-7. The key is 18 inches deep, as detailed in Figure 9-15.

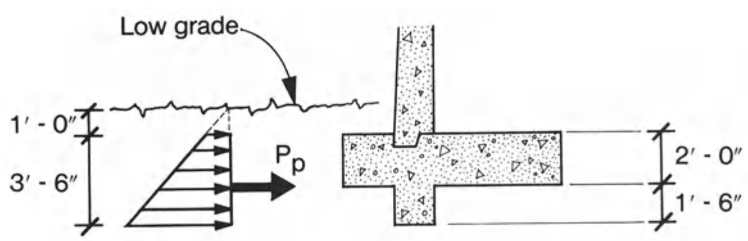


FIGURE 9-15.

The passive pressure computed from Formula (9-4) is:

$$P_p = 2.77 \times 115 \times 3.5\,(4.5 - 3.5/2) = 3066\ \#/\text{ft}$$

Combining this passive pressure with the 7244 #/ft due to friction, as determined in Example 9-7, the safety factory against slide, is:

$$SF = \frac{3066 + 7244}{5075} = 2.03$$

The addition of this shear key increased the safety factory substantially, and this design should be considered satisfactory.

Example 9-9

Required: To determine the effect that a prestressed tiedown cable would have on the resistance to slide for the wall of Example 9-7.

Assume a post-tensioned cable arrangement, as indicated in Figure 8-17. From other sources it has been determined that the cable is a 7 wire strand, having the following properties:

$$A_{ps} = 0.153 \quad \text{and} \quad f_{pu} = 250 \text{ ksi}$$

The ultimate strength of the cable is:

$$P_{\text{ult}} = 0.153 \times 250 = 38.3 \text{ kips}$$

And the effective prestress is:

$$P_{\text{eff}} = 0.70 \times 38.3 \times (1.00 - 0.15) = 22.7 \text{ kips}$$

The 0.70 represents the ratio of allowable stress to ultimate stress, after anchorage of the cable. The 0.15 accounts for the expected long term loss in percentage of prestress.

Destruction tests, in accordance with Section 8-10, were made on several representative cable installations. In each test it was determined that the cable failed at its approximate ultimate strength, and there was no apparent failure in the grout anchorage. It is reasonable, therefore, to use the full effective prestress of 22.7 kips.

Assuming the cable to be at a 30° angle, measured from the vertical, the force components are as given in Figure 9-16. Assuming that the cables are placed 5 foot on center along the wall, then:

$$H = 2270 \quad \text{and} \quad V = 3932 \text{ \#/ft}$$

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.

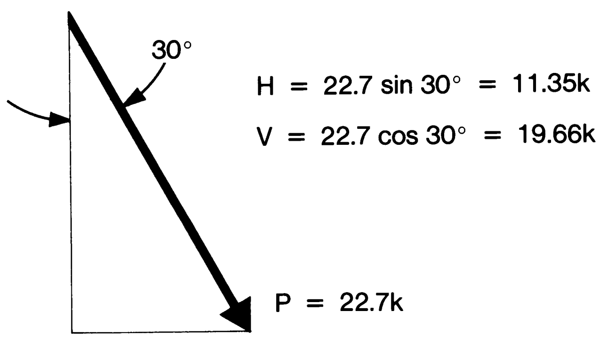


FIGURE 9-16.

The additional resistance to slide on a per foot basis is equal to:

$$2270 + 3932 \tan \phi$$

If this effect were applied to Example 9-7, the total resistance to slide would be:

$$8518 + 2270 + 3932 \tan 28^\circ = 12878 \text{ \#/ft}$$

with a resulting safety factor of:

$$SF = \frac{12878}{5075} = 2.53$$

Note, also, that the vertical component of this cable will have the effect of increasing the contact pressures p_a and p_b, as computed in Example 9-6.

Example 9-10

Required: To compute the design moment for the stem of the cantilever retaining wall originally introduced in Example 9-5. Refer to Figure 9-17 for loading.

$$p_1 = 0.31 \times 400 = 124 \text{ psf} \qquad P_1 = 124 \times 12 = 1488 \text{ \#/ft}$$

$$p_2 = 0.31 \times 110 \times 12 = 409 \text{ psf} \qquad P_2 = \tfrac{1}{2} \times 409 \times 12 = 2454 \text{ \#/ft}$$

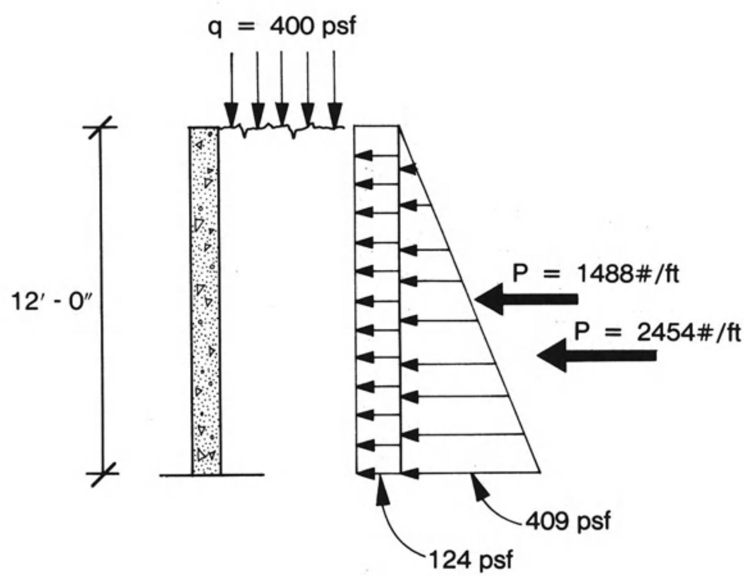


FIGURE 9-17.

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 9-11

Required: To determine the moments induced into the footing of the wall examined in Example 9-5. Refer to Figure 9-18 for details and computations.

The total load on the heel of the footing (9050 #/ft) is the sum of the loads for items 1 and 2, as given previously in Table 9-4.

Pressures at points *a*, *b*, *c*, and *d*:

$$p_a = 2944 \text{ \#/ft} \quad \text{from Example 9-6}$$

$$p_b = (2944 - 256)\frac{6.5}{8.5} + 256 = 2312 \text{ \#/ft}$$

$$p_c = (2944 - 256)\frac{5.25}{8.50} + 256 = 1916 \text{ \#/ft}$$

$$p_d = 256 \text{ \#/ft} \quad \text{from Example 9-6}$$

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

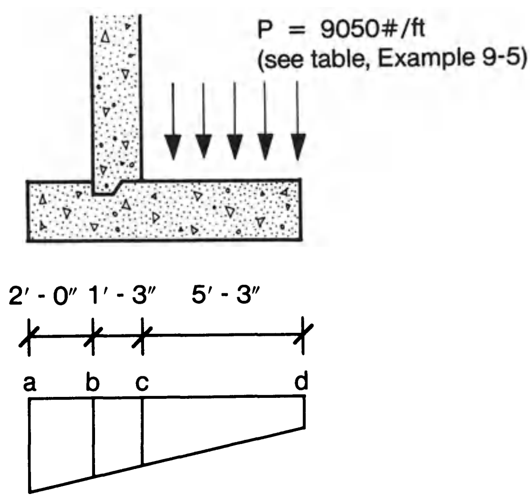


FIGURE 9-18.

moment produces tension on the bottom face of the footing. Note that the weight of earth above the footing is ignored:

$$M_b = 2312 \times \tfrac{1}{2} \times 2.0^2 + (2944 - 2312) \times \tfrac{1}{2} \times 2.0 \times \tfrac{1}{3} \times 2.0 - 300 \times \tfrac{1}{2} \times 2.0^2$$
$$= 4867 \text{ ft\#/ft}$$

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.

$$M_c = (400 + 110 \times 12 + 300)\tfrac{1}{2} \times 5.25^2 - 256 \times \tfrac{1}{2} \times 5.25^2 - (1916 - 256)\tfrac{1}{2} \times 5.25 \times \tfrac{1}{3} \times 5.25$$
$$= 16{,}684 \text{ ft \#/ft}$$

10

Soil Compaction

10-1. GENERAL

Soil compaction is a process whereby a layer of soil is densified by forcing air out of the voids. The effects of soil compaction are as follows:

1. The thickness of the layer (or total volume) is slightly decreased.
2. There can be a significant increase in unit weight (density).
3. There is a corresponding increase in shear strength.
4. The settlement of the compacted soil will be substantially less than that of the uncompacted soil.

All of the above effects are good, and the net result is to increase the capability of the soil to support load. Compaction will also decrease the permeability of the soil, which may good or bad, depending upon usage and the desired result.

Compaction is sometimes confused with consolidation, since the soil is densified in both processes. *Compaction,* as noted previously, is the term which indicates the densification of a soil by forcing air out of the voids. Compaction is the result of a man-made effort performed either in the laboratory or in the field, and is a rather quick process, being measured in minutes or hours. *Consolidation,* in contrast, is the term used to indicate the densification of a soil by forcing water out of the voids. 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 a maximum thickness of approximately eight inches. 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. It is desirable, and frequently mandatory, that there should be no planes of weakness within the mass. 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 insure 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 input regarding the technical and performance areas of the work before committing himself to any course of action.

10-2. BORROW FILL

Borrow fill is a term used to identify soil which must be brought from its natural place to the place where it will be used. Borrow fill may be required because of any one of the following conditions:

1. When the existing grade must be raised to meet architectural design requirements
2. To replace soil having unsatisfactory bearing capacity with stronger soil
3. For use as backfill against walls and in areas of overexcavation

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 which are proper for the intended use should be considered, but this decision must also be influenced by the cost effectiveness and availability of the different types of soil.

An examination of the soil groups listed in Table 10-3 (see Section 10-11) provides insight as to the characteristics of those soils from considerations of density, compaction, stability, bearing capacity, settlement, and expansion. Soil groups designated GW, GP, GM, SW, SP, and SM are the best from the standpoint of quality assurance during compaction, and can be expected to produce a well densified soil mass. In those instances when the borrow fill must support critical loads, it should be noted that the SP and SM soil groups are suspect. For all around excellence as a borrow fill, the architect should limit his selection of material to those included in soil groups GW, GP, GM, and SW.

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 10-1 shows the soil being loaded at the borrow pit for transport to the site of the building.

FIGURE 10-1. Soil being loaded at a borrow pit. [Ref. 7]

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.

10-3. SITUATIONS WHERE SOIL COMPACTION IS REQUIRED

Compaction serves a necessary function in almost all architectural engineering projects, occurring 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 this layer 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. There could be a combination of items 1 and 2, in which case the thin layer of poor soil would have to be removed before building up the grade with compacted borrow fill.
4. Backfill placed against basement walls and retaining walls must meet certain materials specifications and compaction requirements. These requirements are discussed in detail in Chapter 8.
5. Earth is sometimes 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.
6. General areas of overexcavation, either in width or in depth, and areas adjacent to footings, where side walls collapse or spall off before concrete is poured. This is a particular problem where wall footings are stepped, as in Figure 5-7. These latter areas may be filled with footing concrete rather than being backfilled 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, land fill, and soil reclamation.

10-4. 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, 2, and 3 of Section 10-3. 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 previously established for the particular soil being compacted. The spreading out of soil in layers is illustrated in Figure 10-2.

Prior to the start of compaction, the moisture content of any soil containing a significant amount of cohesive material (borrow fill or natural earth, as the case may be) must first be adjusted until the specified moisture content has been achieved. This may require several adjustments. The moisture content must also be determined anew after each adjustment, and compared to that which has been specified. This whole process can be quite time consuming.

FIGURE 10-2. Spreading soil before compaction. [Ref. 7]

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 over the area under its own power. Other machinery is not motorized, and must be towed. All of this machinery, however, is designed to cover large areas of soil quickly, and to exert very large contact pressures. The compacting elements consist of heavy rollers, which may be pneumatic, smooth drum, or paddlefoot. Smooth drum and paddlefoot are frequently referred to as steel-wheeled and sheepsfoot, respectively. Rollers may also be mechanically vibrated in order to increase their effectiveness in compaction. Several common types are illustrated in Figure 10-3.

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 Table 10-3 of Section 10-11.

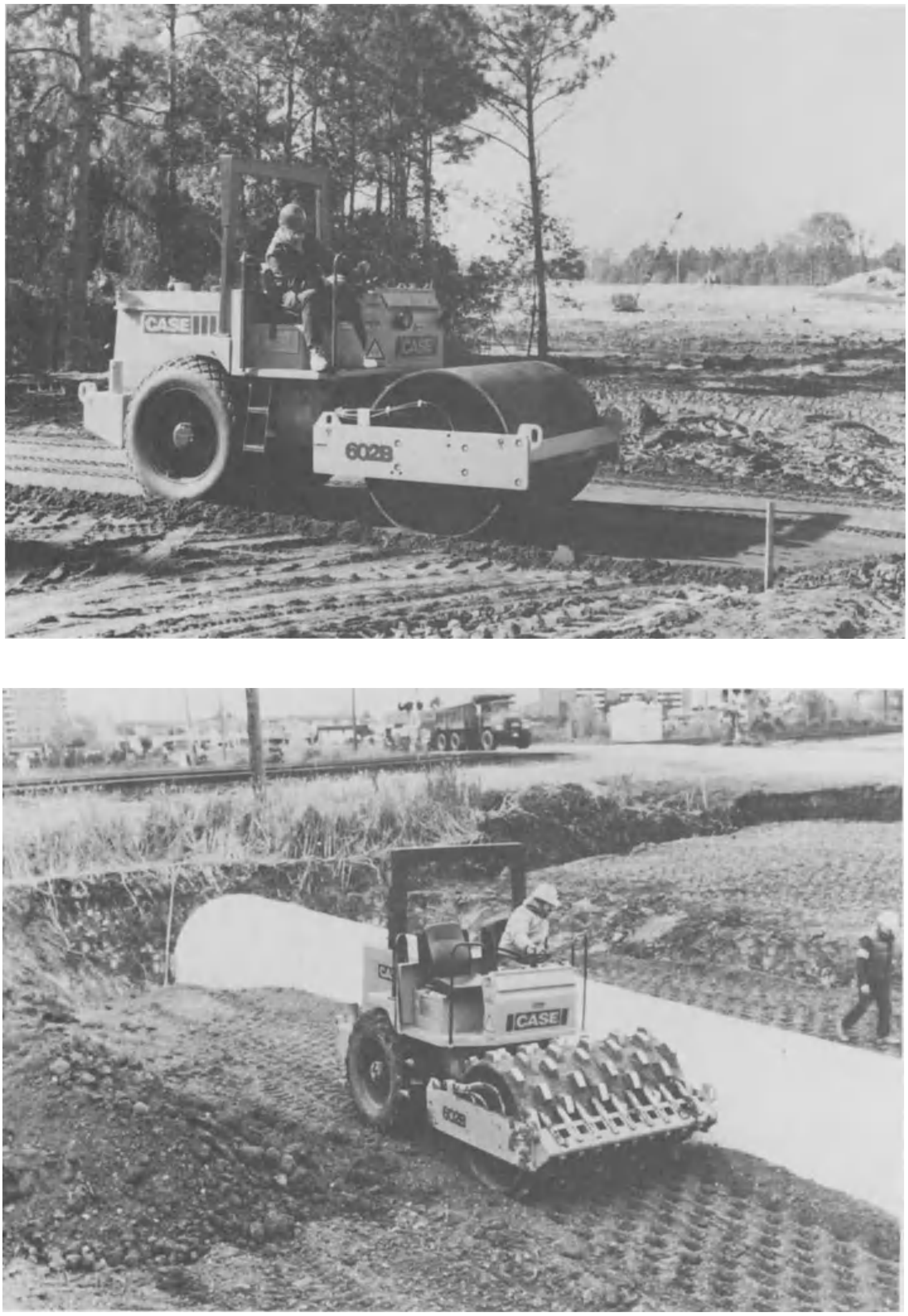


FIGURE 10-3. Different types of machine driven vibratory compactors. (a) smooth drum roller, (b) paddlefoot drum roller. [Ref. 7]

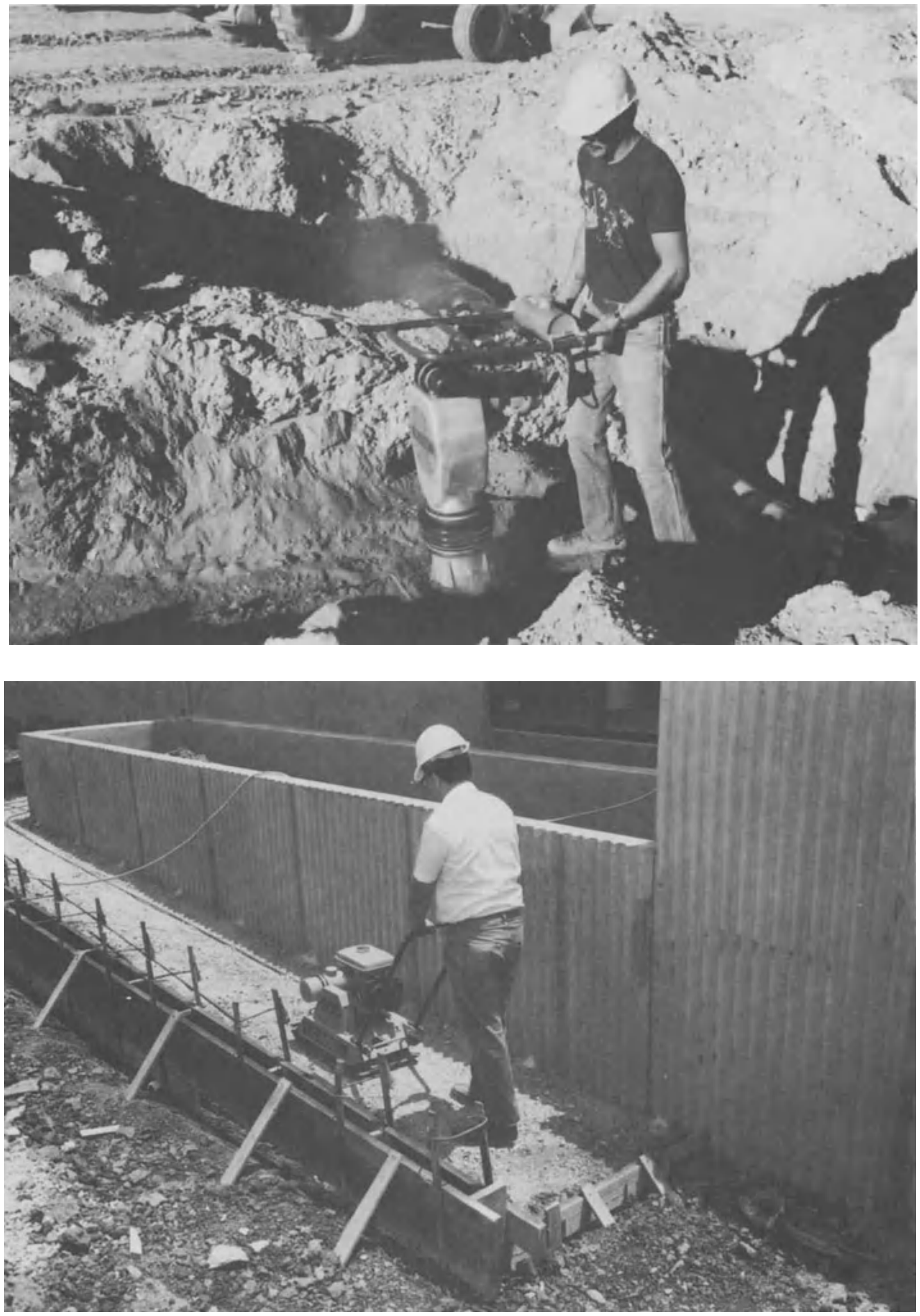


FIGURE 10-4. Different types of hand operated vibratory compactors. (a) rammer, (b) self-propelled plate compactor. [Ref. 7]

10-5. COMPACTION OF SMALL, CONFINED AREAS

Small, confined areas occur where walls must be backfilled, or where there has been intended or unintended overexcavation, such as those described in items 4, 5, and 6 of Section 10-3. These areas require great care in the depositing of the soil so as to not damage adjacent construction. This work can sometimes be done with a front end loader, but at other times must be done by hand.

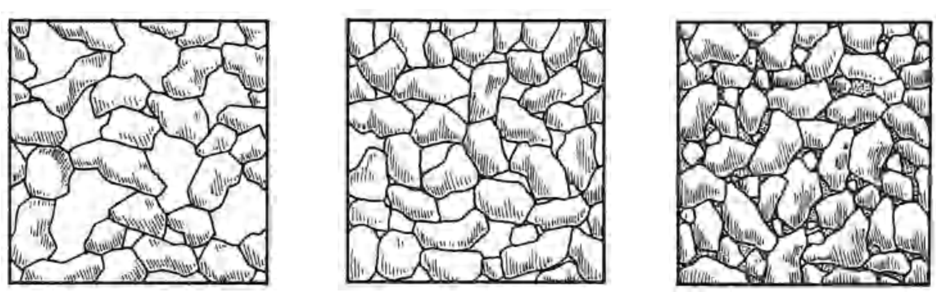
Compaction in these small areas follows the same basic procedures as those described for large areas. This includes the determination of optimum layer thickness, moisture control, and in-situ density tests. The main difference in performing the work in small areas is that the work must be done by hand. There are two basic kinds of portable compaction equipment available for that purpose. One is a hand-held rammer, the other is a self-propelled vibratory compactor. Both kinds are illustrated in Figure 10-4.

10-6. 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 10-5(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



(a) (b) (c)

FIGURE 10-5. Intergranular seating and gradation of particles: (a) poorly graded, poorly seated particles; (b) poorly graded, but well seated particles; (c) well graded and well seated particles.

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 10-5(b).

The arrangement of particles shown in Figure 10-5(b) 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, in which case there is 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 10-5(c) not only provides maximum intergranular contact, but also provides inherent stability. This very important property 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 curve in Figure 1-3 in Chapter 1, 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 in that they may separate the larger grains, thereby 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 of these separate actions increase the 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 usually considered to be adequate 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 numerically compare 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 10-1 are generally considered to be adequate.

TABLE 10-1. Recommended Minimum Relative Density of Soil.

D_r, %	Location
85–90	Backfill against basement walls and retaining walls which 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: The values contained in Table 10-1 are guidelines only. The values to be used on any project must be determined by the architect or engineer in responsible charge of the work.

Determination of Relative Density

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 has been described in Section 2-8. Void ratios must first be determined for each of the loosest, in-place, and densest states. This is done by performing Steps 1 through 6 of Section 2-1, the results of which are used to determine void ratio values by Formula (2-3). The relative density is then computed by Formula (2-11). Example 2-3 illustrates the calculations required for this method.

Method 2. Unit weight. Relative density can be expressed in terms of unit weight, as follows:

$$D_r = \frac{\gamma_{\mathrm{nat}} - \gamma_{\mathrm{min}}}{\gamma_{\mathrm{max}} - \gamma_{\mathrm{min}}} \times \frac{\gamma_{\mathrm{max}}}{\gamma_{\mathrm{nat}}} \times 100\% \tag{10-1}$$

Where:

γ_{max} is the dry unit weight of the soil in its densest state,
γ_{nat} is the dry unit weight of the soil in place, and
γ_{min} is 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 because there is a 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 (10-1) is the result of transforming Formula (2-11) by substituting unit weight equivalents for void ratios. These equivalents are determined as follows.

First, using Formulas (2-1), (2-8), and (2-5):

$$\gamma = \frac{W_w + W_s}{V}, \quad \text{but} \quad W_w = 0, \quad \text{therefore} \quad \gamma = \frac{W_s}{V}$$

Then:

$$\gamma = \frac{62.4\, G_s V_s}{V} = \frac{62.4 G_s}{V} \times \frac{V}{e+1} = \frac{62.4 G_s}{e+1}$$

And:

$$e = \frac{62.4\, G_s}{\gamma} - 1 = \frac{62.4 G_s - \gamma}{\gamma}$$

Finally, using proper subscripts, develop three void ratio expressions in terms of unit weight, then substitute these expressions into Formula (2-11) and perform the necessary algebra. For example:

$$e_{\text{nat}} = \frac{62.4 G_s - \gamma_{\text{nat}}}{\gamma_{\text{nat}}}$$

Remember, when making these substitutions, that e_{max} corresponds to γ_{min} and e_{min} corresponds to γ_{max}.

10-7. COMPACTION OF FINE GRAINED SOILS

General

The theory by which compaction works for a fine grained soil is entirely different than 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 a blanket of water, the molecules of which 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 detailed discussion of this phenomenon refer to Chapter 11, the subject of which is expansive clays.

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 which 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 10-2.

TABLE 10-2. 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 of Testing Materials has adopted this procedure, with the following identification:

ASTM Test Method 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, 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 as D-698, 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, the lower part being called the *mold* and the upper part being 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—a 4 inch mold having a volume of 1/30 CF, the other a 6 inch mold having a volume of 0.075 CF. The 4 inch diameter mold is detailed in Figure 10-6.

There are four methods by which this test can be performed. Except for the choice between Methods A and B, the choice of methods depends upon grain size and distribution within the soil.

Method A: 4 inch mold. Material retained on a No. 4 sieve is disgarded. 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 disgarded. When the amount of material retained is 10% or greater, then Method D should be used instead.
Method D: 6 inch mold. Material retained on the 3/4 inch sieve is placed on a 3

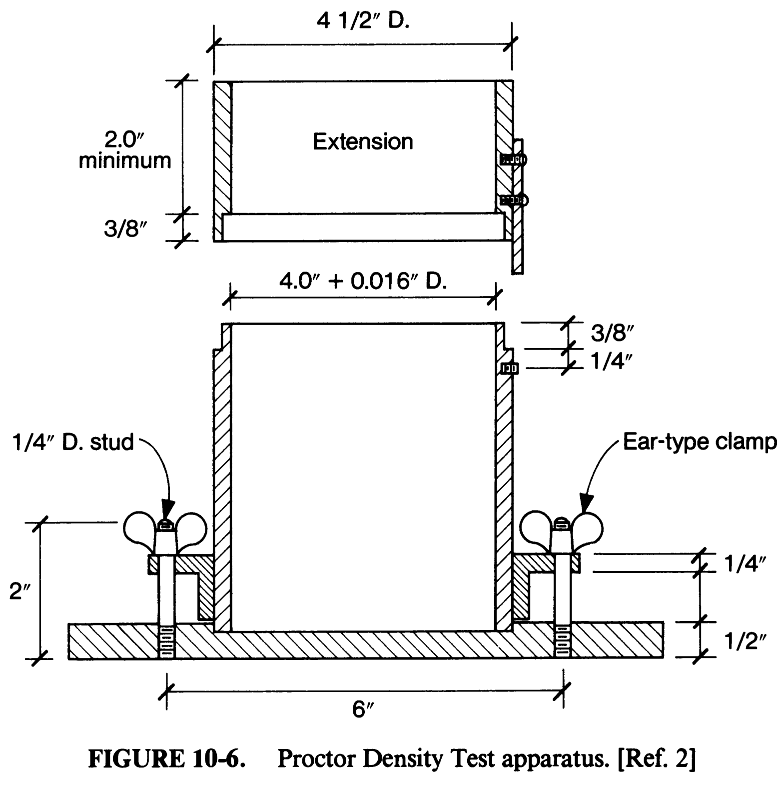


FIGURE 10-6. Proctor Density Test apparatus. [Ref. 2]

inch sieve. Material retained on the 3 inch sieve is disgarded. 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.

Prior to performing the test a sufficient quantity shall be obtained of representative soil for testing. The moisture content of all material shall then be reduced until the material can be easily crumbled between the fingers; it is not, however, required that all water be removed at this stage.

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 insure 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 in the case of the 4 inch mold, and 56 blows in the case of the 6 inch mold.

After compaction remove the collar and trim the sample even with the top of the mold. Compute the compacted, wet weight of the sample. Then determine the dry weight of the same sample in accordance with the following ASTM Standard:

ASTM Test Method D-2216: Laboratory Determination of Water (Moisture) Content of Soil, Rock and Soil-Aggregate Moistures

Numerical values for density and moisture content are now computed by use of the following formulas:

$$\text{wet unit.df.weight} = \gamma_{\text{nat}} = \frac{W}{V} = \frac{W_w + W_s}{V} \tag{10-2}$$

$$\text{moisture content} = w\% = \frac{W_w}{W_s} \times 100\% \tag{10-3}$$

$$\text{dry unit weight} = \gamma_{\text{dry}} = \frac{W_s}{V} \tag{10-4}$$

$$\gamma_{\text{dry}} = \frac{\gamma_{\text{nat}}}{w + 1} \quad (w \text{ in decimal}) \tag{10-5}$$

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 10-7.

Point 1 is called the optimum moisture content because this is the moisture content at which maximum compaction, and hence maximum density, can be

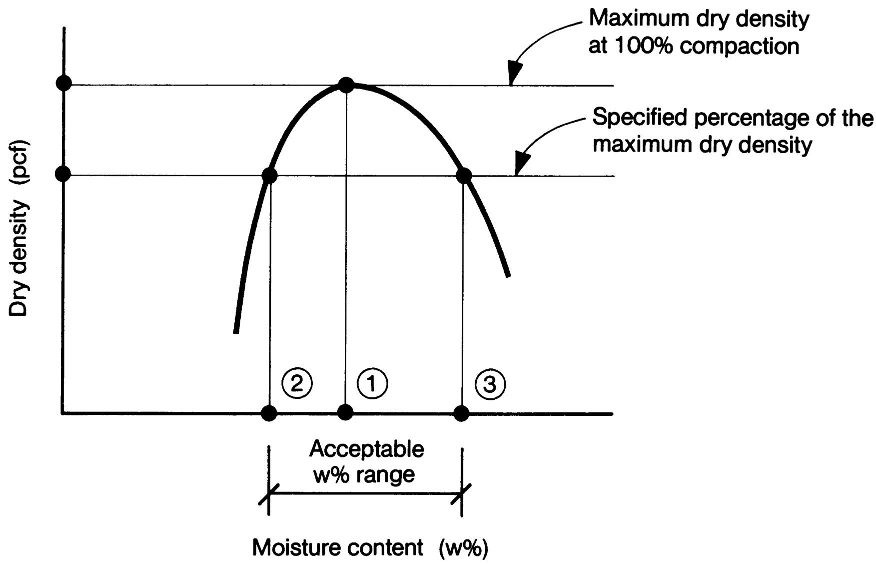


FIGURE 10-7. Curve showing variation in the dry density of a soil as a function of moisture content.

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.

10-8. 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 which are mixed grained will in all likelihood exhibit some of the characteristics of both coarse grained and fine grained soils. The deciding factor as to whether a particular soil should be compacted in accordance with coarse grained or fine grained requirements is that of cohesion.

1. Soils which do not exhibit any measurable cohesion: Treat as coarse grained soil—base compaction on the relative density D_r.
2. Soils which do exhibit measurable cohesion: Treat as fine grained soil—base compaction on the Proctor Density Test.

10-9. VERIFICATION OF IN-PLACE 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. For coarse grained soil: Specify the required minimum relative density. This value can be selected from those given in Table 10-1, or can be based on the designer's own experience.
2. For fine grained soil: Specify the required minimum dry density. Then determine the acceptable range of moisture content through which this density can be achieved.

For both classifications of soil, the dry density of the in-place, compacted soil must be determined. Undisturbed samples of soil could be set to the laboratory for analysis. This procedure, however, would take time and the field work could be delayed while waiting for test results and subsequent approval or correction of the work. There are three procedures whereby the wet density of the in-place soil can be readily determined in the field. Once the in-place density and the moisture content are known, the dry density can be easily computed. These procedures are described in the following ASTM Standards:

ASTM Test Method D-1556: Density of Soil in Place by the Sand-Cone Method.

This method is generally limited to soil in an unsaturated condition. It is not recommended for soil that is soft or easily crumbed, or for deposits where water will seep into the test hole.

ASTM Test Method D-2167: Density and Unit Weight of Soil in Place by the Rubber Balloon Method. (This method is illustrated in Figure 10-8.)

This method is not suitable for use with organic soils, or soils that are saturated, or soils that are highly plastic. The use of this method will require special care with unbonded granular soils, soils containing appreciable amounts of coarse aggregate larger than $1\frac{1}{2}$ inches, granular soils having a high void ratio, and fill materials having particles with sharp edges.

ASTM Test Method D-2922: Density of Soil and Soil-Aggregate in Place by Nuclear Methods. (This method is illustrated in Figure 10-9.)

This method provides a rapid, nondestructive technique for the determination of in-place wet soil density. Test results may be affected by chemical composition, heterogeneity and surface 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 hazzards and

FIGURE 10-8. Soil density by the rubber balloon method. [Ref. 17]

FIGURE 10-9. Soil density using nuclear methods. [Ref. 4]

require special care. The work must be done in strict conformance with all safety requirements and must be performed only be trained personnel.

It should be noted that regardless of which of the three methods are used to determination the in-place wet density, it must be remembered that it is the dry density that is ultimately required. This latter density can easily be computed once the moisture content of the soil is known. Refer to Section 10-10 for discussion of two different procedures.

10-10. FIELD CONTROL OF MOISTURE CONTENT

There are two general procedures whereby moisture content can be determined:

1. Accurate results can be achieved by the laboratory analysis of undisturbed samples in accordance with the previously noted ASTM Test Method D-2216. 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.

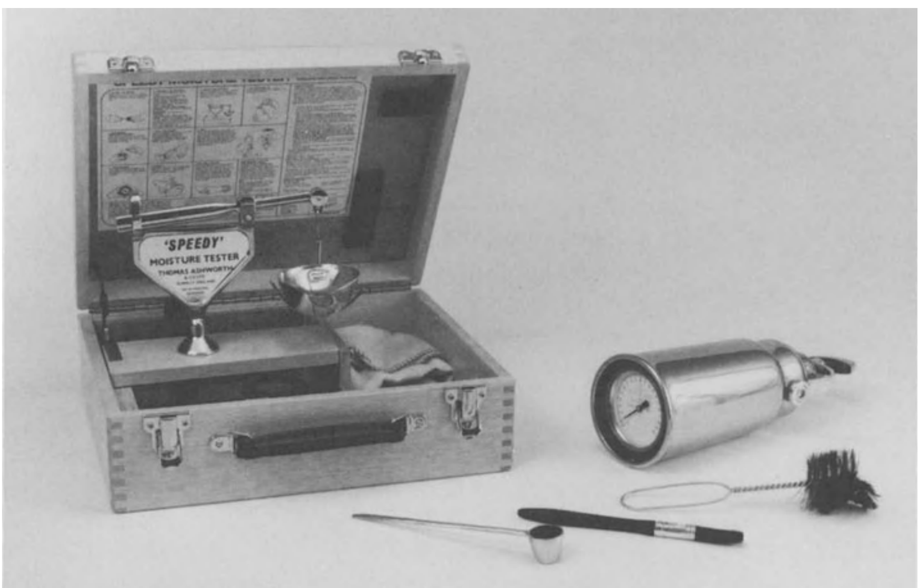


FIGURE 10-10. Speedy Moisture Tester for determination of moisture content of soil in the field. [Ref. 4]

2. Very quick results can be obtained in the field with a portable moisture tester, as illustrated in Figure 10-10. This particular tester, which conforms to AASHTO Designation T-217, provides for almost continuous monitoring of the moisture content because the test can usually be performed in three minutes or less.

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, after which it must be thoroughly mixed 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.

10-11. COMPACTION CHARACTERISTICS OF USCS SOIL GROUPS

Table 10-3 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 six following items.

1. *USCS Symbol:* Soil groupings in accordance with the Unified Soils Classification System, refer to Table 1-3 in Chapter 1.
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 proven 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-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.

TABLE 10-3. 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

TABLE 10-3. (*Continued*)

1	2	3	4	5	6
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				

10-12. SAMPLE PROBLEMS

Example 10-1

Required: To determine the relative density of the soil previously analysed in Example 2-3 by using the procedure specified in Chapter 10.

The following weights and measures where determined in Examples 2-1 and 2-3.

	Loosely Packed	In Place	Tightly Packed
Step 1. V	1 CF	1 CF	1 CF
Step 2. W	97.8 #	117.6 #	126.6 #
Step 3. W_s	87.6 #	105.0 #	113.3 #

Using dry density values, from Formula (10-1),

$$D_r = \frac{105.0 - 87.6}{113.3 - 87.6} \times \frac{113.3}{105.0} \times 100\% = 73.0\%$$

Note: This answer agrees exactly with that previously determined in Example 2-3 with the use of void ratios.

If in-place density values were used:

$$D_r = \frac{117.6 - 97.8}{126.6 - 97.8} \times \frac{126.6}{117.6} \times 100\% = 74.0\%$$

It is because of this slight discrepancy that D_r is always computed using dry density values.

Example 10-2

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

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 (10-2):

$$\gamma_{nat} = \frac{4.07}{1/30} = 122.1 \text{ pcf}$$

From Formula (10-3):

$$w = \frac{0.54}{3.53} \times 100\% = 15.3\%$$

From Formula (10-4):

$$\gamma_{dry} = \frac{3.53}{1/30} = 105.9 \text{ pcf}$$

or from (10-5):

$$\gamma_{dry} = \frac{122.1}{0.153 + 1} = 105.9 \text{ pcf}$$

Example 10-3

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.

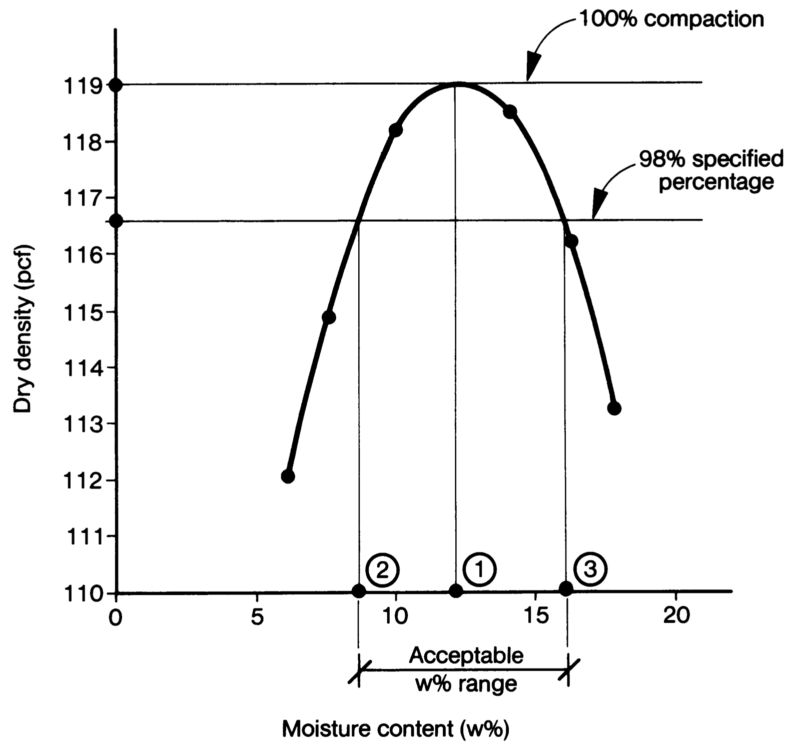


FIGURE 10-11.

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 10-11. The following information is then read from the curve:

1. Maximum density is 119 pcf at 100% compaction
2. Corresponding moisture content is 12.1%
3. The acceptable range of moisture content for 98% compaction is 8.7 to 16.1%

11

Expansive Clay

11-1. 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,* or 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 an 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, 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 on the particle's surface. Extensive research has determined that colloidal particles consist primarily of clay minerals.

11-2. CLAY MINERALS

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 diagrammatically illustrated in Figure 11-1.

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.

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 which is 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 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 which is contained within the voids of the soil. It is frequently referred to as *pore water.*

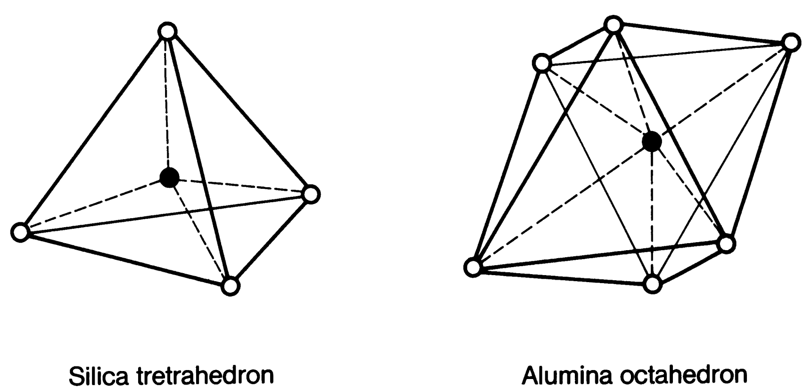


FIGURE 11-1. The basic building blocks of clay minerals, in which the open circles indicate oxygen atoms (O) or hydroxyl ions (OH), and filled circles indicate a silica or aluminum atom, or a replacement atom.

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 surfaces. 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-grained 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.

11-3. 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 — $Al_4Si_4O_{10}(OH)_8$

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 principal mineral. Instead, this name refers to a group of micalike 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, and has little tendency to change volume when exposed to a change in moisture content unless there is a deficiency in potassium, in which case the illite particle will exhibit an increased tendency for volume change.

Illites are used extensively in the manufacture of brick and tile.

Montmorillonite — $Si_8Al_4O_{20}(OH)_4 \cdot nH_2O$

Montmorillonite is a group name for clay minerals which have expansive structures, and is also the name of the principal 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 existence 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, thereby 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.

A similar procedure is occasionally used when test borings are drilled through collapsible material. One of the problems associated with this procedure is the recovery of undisturbed soil samples.

11-4. 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, which are 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.*

11-5. 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.

$$1 \text{ millimeter (mm)} = 0.001 \text{ meter m}$$

$$1 \text{ micron } \mu = 0.001 \text{ mm}$$

$$1 \text{ nanometer (m}\mu) = 0.001\ \mu = 0.000\ 001 \text{ mm}$$

$$1 \text{ angstrom Å} = 0.1 \text{ m}\mu = 0.0001\ \mu = 0.000,\ 000\ 1 \text{ mm}$$

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.

11-6. 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 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 sample 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,* and is 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 Test Method D-4318 for Liquid Limit, Plastic Limit and Plasticity Index for Soils

The type of equipment used in determining the liquid limit is shown in Figure 11-2.

Plastic Limit *(PL)*

After the sample has entered the plastic state it can be rolled into long, thin threads, and when suspended from the fingers it 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. The equipment used in performing this test, however, is somewhat different, as shown in Figure 11-3.

FIGURE 11-2. Liquid limit testing set. [Ref. 4]

FIGURE 11-3. Plastic limit testing set. [Ref. 4]

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 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 Test Method 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 11-1.

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 11-2.

It should be noted that soils which exhibit change in volume with change in water content are usually referred to as *swelling soils.* Remember, however, that 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.

TABLE 11-1. Atterberg Limits for Remolded Clay.

Consistency	Physical State	Atterberg Limits
Very soft	Liquid	
Soft	———	Liquid limit *LL*
Stiff	Plastic	*PI*
Very stiff	———	Plastic limit *PL*
Hard	Semi-solid	
	———	Shrinkage limit *SL*
	Solid	

TABLE 11-2. Plasticity Index Evaluation.

Plasticity Index *PI*	Swelling Potential	Percent Increase
0	Non-plastic	—
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

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. An in-situ 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.

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 things:

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.*

11-7. 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 11-2 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 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 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 build up 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.

11-8. REASONS WHY WATER CONTENT CHANGES

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 draw more water from the ground than 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.

Unforeseen Problems

11. Leaks in underground piping.

11-9. BUILDING CONSTRUCTION

General

This section 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 never be constructed on soil whose plasticity index is 10 or greater, 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 protected elements, such as interior slab on ground and exterior paving, to be constructed on an expansive soil, subject to its load bearing characteristics. The fact that a soil is an expansive soil is not, in itself, the 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 this soil is confined to a relatively thin layer, and when there is good bearing soil at a reasonable depth beneath it, then 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 can not 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. Alternative methods are discussed in detail in another section of the text.

Construction of Slab on Ground

There are several different types of slab on ground installations, each of which must be considered separately.

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 twenty feet below the exterior grade. In the majority of cases, therefore, the basement slab will be well below the zone of expansive soil. The slab, then, can be designed as a normal slab on ground, in accordance with the recommendations in Appendix D.

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. There are several methods by which this may be done.

The first and obvious solution to eliminating this problem is to eliminate the offending soil. When this soil occurs only in a relatively thin layer, and is accessible to heavy earth moving machinery, then that layer of soil may be easily and cost effectively removed and replaced with acceptable borrow fill. 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 other methods must be used.

In the second method the beams and slabs are poured directly on hollow cardboard forms which are manufactured especially for that particular purpose. The use of these forms is illustrated in Figure 11-4.

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. Because this floor, therefore, must eventually be self supporting, it must be designed and constructed as a typical reinforced concrete system of slabs, beams, piers, and foundations.

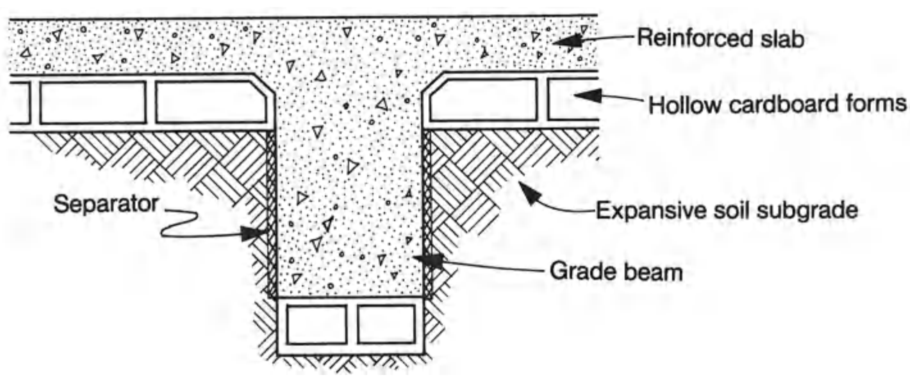


FIGURE 11-4. Construction of slab on ground with hollow cardboard forms.

This method of construction has several inherent problems. These include:

1. Maintaining the integrity of the surface of the subgrade prior to and during installation of the cardboard forms
2. Installation of reinforcing, and maintaining it in proper position prior to and during the pour
3. 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 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.

The third method, which may well be preferred from the standpoint of assured structural integrity, is to create a crawl space or plenum beneath the slab. This alternative has the additional advantage of simplifying installation and future access to many of 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.

11-10. RESIDENTIAL CONSTRUCTION

Residential construction, unfortunately, for reasons of time or money or ignorance, is rarely given the thoughtful consideration that should be required for any 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 beneath the layer of the offending soil.
2. Migration of moisture will not normally occur at the depth necessary to cause any appreciable movement in a basement slab.

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 shown in Figure 11-5.

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. Residences in temperate climates, therefore, are commonly constructed with the first floor slab poured directly on the ground. The type of construction generally used is indicated in Figure 11-6.

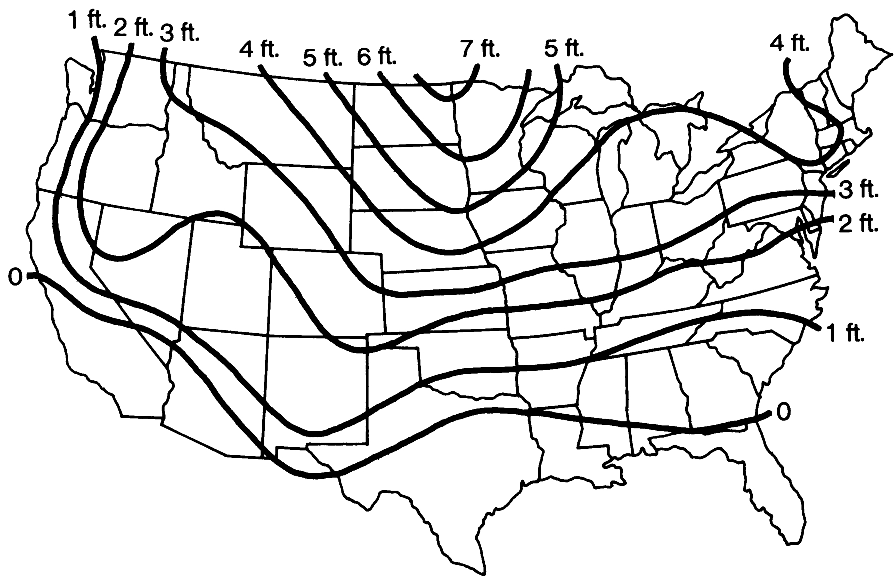


FIGURE 11-5. Maximum anticipated depths of freezing as inferred from city building codes. Actual depths may vary considerably depending on cover, soil, soil moisture, topography, and weather. [Ref. 18]

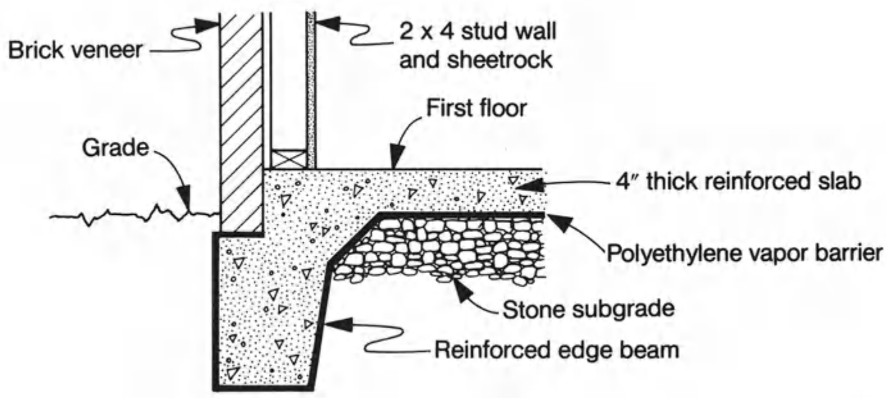


FIGURE 11-6. Typical edge beam detail for a residential slab on ground.

11-11. THE EFFECT OF MOISTURE CHANGE IN RESIDENTIAL CONSTRUCTION

Prior to 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 completed. 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 11-7.

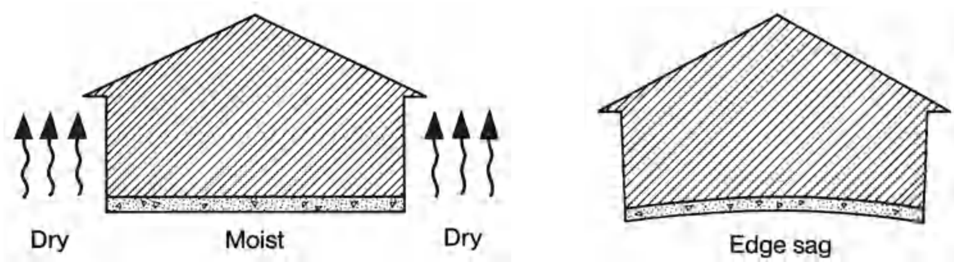


FIGURE 11-7. Residential edge sag, due to an extended period of dry weather.

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 11-8.

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 runoff are rarely the same around the entire perimeter of a residence. Sections of perimeter may shrink or swell differently than other sections. Differential lift or sag around the perimeter of a residence can be disastrous, as illustrated in Figure 11-9.

This process of gain or loss of 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 an 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

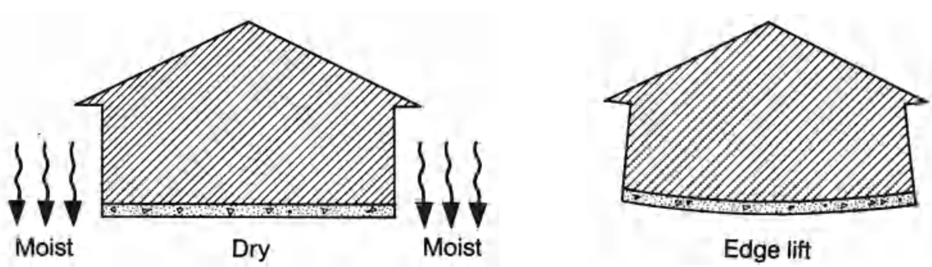


FIGURE 11-8. Residential edge lift, due to an extended period of wet weather.

FIGURE 11-9. Damage to a residence, caused by differential settlement.

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.

11-12. THE EFFECT OF MOISTURE MIGRATION IN RESIDENTIAL CONSTRUCTION

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:

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 wet area to a dry area, never from dry to wet.

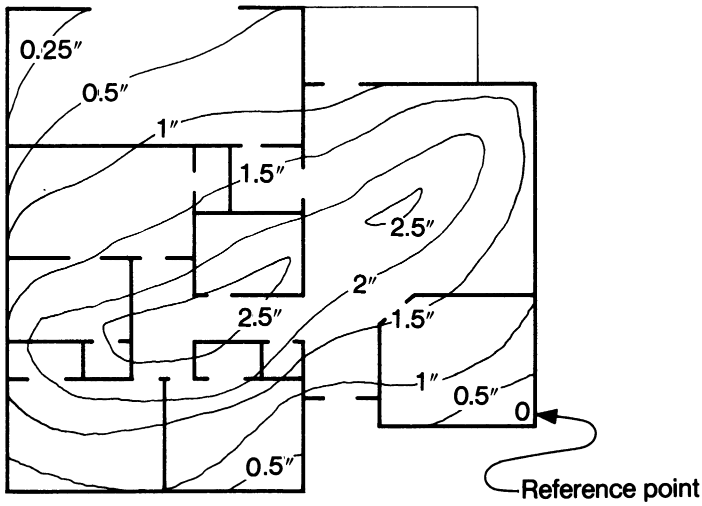


FIGURE 11-10. Variation in slab topography as caused by the migration of moisture beneath the slab.

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 six inches. An example of the type of variation that can be found in topography of a typical residence is shown in Figure 11-10.

11-13. GENERAL RECOMMENDATIONS REGARDING RESIDENTIAL CONSTRUCTION

The proper way to construct the foundations and slab on ground for a house is to follow the procedure previously outlined for building construction. Anything else is a compromise. The following compromises are listed not as a means of eliminating the effects of a shrinking or swelling soil but to attempt to limit them so as to avoid excessive damage.

1. Footings for all load bearing walls should extend to the greater depth as determined by: (a) 24 inches below finished grade or (b) 18 inches below natural grade.
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 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. An alternative solution to the stabilization of subgrade is that of soil stabilization. This may be accomplished by the addition of hydrated lime, $Ca(OH)_2$. This alternative 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 it is used improperly. For additional information on the use of lime as soil stabilization see Krebs and Walker, 1971 [Ref. 10], p. 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, then 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. The thought behind this kind of 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 6 inches is recommended. Although post-tensioning can produce good results, the author has observed a number of house builders who have no idea of the complexities of this type of construction, so that the work is simply not done properly. The advantages that post-tensioning could produce, therefore, are rarely realized.

It has been the author's experience that foundations built on expansive soil sag 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.

11-14. 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's level to check whether walls are out of plumb, particularly corners.
5. Look for cracks in brickwork or in plaster, particularly those which 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.

11-15. 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 hold free water. (Section 11-2.) During the excavation the pressures which previously had maintained equilibrium were released. Free water then 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 the status quo is changed in any way.

12

Characteristics of Rock

12-1. GENERAL

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

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.

12-2. 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, as follows:

Igneous: This term applies to rock which 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 which 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

12-3. 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 of these things should be taken into consideration by the designer during the preliminary stage of the project.

12-4. 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.

12-5. 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 and has a diameter of 1³⁄₁₆″ or 2⅛″, depending upon the size core specified. The larger diameter 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 Figure 3-7 of Chapter 3, and in Figure 12-1.

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.

FIGURE 12-1. A typical core box, used to transport rock cores. [Ref. 1]

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 1½ 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 test it is very important that the sample be properly trimmed so as to have both ends cut at right angles to the length of the sample. The numerical value of the unconfined compression strength on rock is usually stated in tons per square foot (tsf) and is symbolized by 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 almost any kind of superimposed lateral or vertical load. In buildings or other structures 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 Section 3-8.

12-6. 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,* symbolized 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 five foot lengths. In computing the RQD, only sound pieces of rock are used; those less than 4″ long are excluded, as are those which are broken or fragmented.

The relationship between the RDQ values and the quality of the in-situ rock is given in Table 12-1.

TABLE 12-1. Relation of RQD and In Situ Rock Quality[16]

RQD %	Rock Quality
90–100	Excellent
75–90	Good
50–75	Fair
25–50	Poor
0–25	Very poor

12-7. ALLOWABLE BEARING PRESSURE

General Considerations

Factors which 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, symbolized by q_u. This property, which is analogous to

the ultimate compression strength of concrete, is determined in accordance with the following ASTM Standard:

ASTM 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 RDQ 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 an inch, the compressibility of the rock mass is reflected by the Rock Quality Designation. Allowable bearing pressures, based on this RQD, are given in Table 12-2.

Bearing Pressure from Code

All building codes contain a section which specifies the allowable bearing pressure that is 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.

TABLE 12-2. Allowable Contact Pressure q_a on Jointed Rock[16]

RQD	q_a (tsf)
100	300
90	200
75	120
50	65
25	30
0	10

Note: q_a shall not exceed q_u

12-8. 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 Building Code of the City of New York, condensed highlights of which are given below:

Allowable Soil Bearing Pressures [Ref. 6]

(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 $\frac{1}{8}$ inch wide, generally no closer than 3 feet apart; core recovery with a double tube, diamond core barrel is generally 85 per cent 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 $\frac{1}{4}$ inch wide and slightly weathered, generally no closer than 2 feet apart; core recovery with a double tube, diamond core barrel is generally 50 per cent 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 per cent 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 inches wide but filled with stiff soil; core recovery with a double tube, diamond core barrel is less than 35 per cent but not less than 20 per cent for each 5 foot run, but standard penetration resistance in soil samples is more than 50 blows per foot—8 tsf.

12-9. 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.

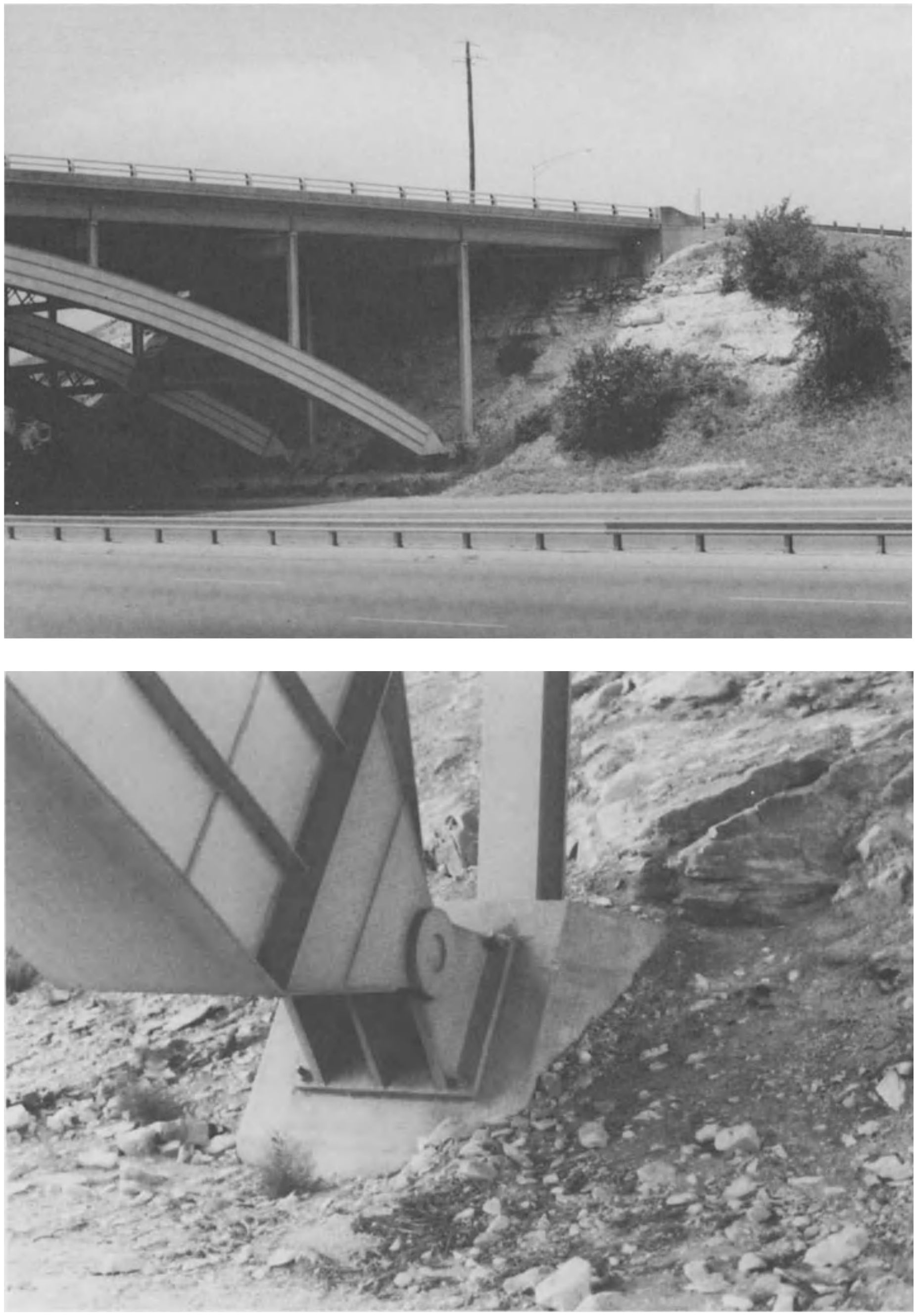


FIGURE 12-2. Arch base bearing on bedrock: (a) overview; (b) close up. [Ref. 21]

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.
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 that is below the water table. The control of water should not imply waterproofing since the flow of water cannot be totally prevented.

12-10. 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 advantage of arch construction, combined with the vertical and lateral load carrying capability of bedrock, has 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 Figure 12-2.

APPENDIX A

Earth Pressure Transfer at Cold Joint by Shear-Friction [Ref. 5]

A-1. ALLOWABLE TRANSFER FORCE

The American Concrete Institute, in section 11.7 and B7.6 of ACI 318-83, specifies a procedure to be applied when it is appropriate to consider the transfer of shear forces across a plane. This concept of transfer is called *shear-friction,* and is applicable to any plane subject to pure shear, as opposed to diagonal tension. Pure shear exists at each of 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 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, therefore, is applicable to both of the above noted conditions.

Slippage along the joint must be prevented because a total shear failure would 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 typical cold joint to which the concept of shear-friction is applicable is illustrated in Figure A-1.

The ultimate shearing force capacity as developed by shear-friction is given in the Code by the following formula:

$$V_n = 0.6\, A_{vf} f_y$$

provided:

1. Concrete is normal weight, 150 pcf.
2. The reinforcing steel is perpendicular to the shear plane.
3. The surface of the shear plane which is poured first shall be clean and free of all laitance.
4. The 0.6 coefficient assumes that the surface poured first has not been intentionally roughened. For those instances when the surface has been roughened to a full amplitude of approximately $\frac{1}{4}$ inch the coefficient may be increased to a value of 1.0.
5. The tensile strength of the reinforcing is developed through adequate embedment length on both sides of the critical section.

This formula can be modified so that the transfer force can be computed directly in terms of actual force, based on working stress, rather than ultimate stress. Modifying the basic formula:

$$V_u = \phi V_n$$

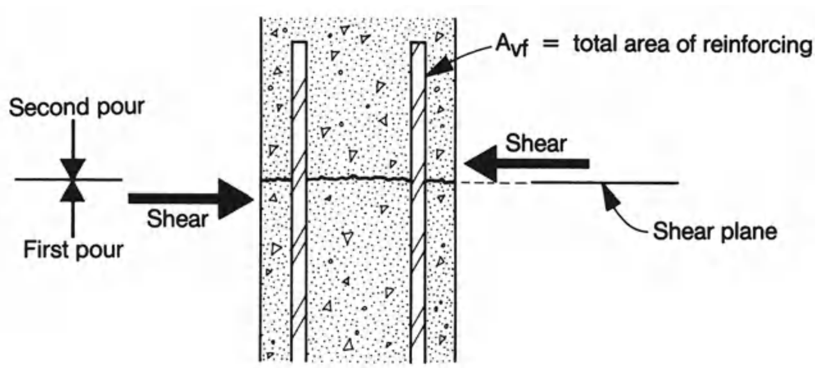


FIGURE A-1. Shear-friction at typical cold joint.

TABLE A-1. Transfer Force in Shear-Friction, per Bar.

Bar Size	Transfer Force
# 3	1980 #
# 4	3600 #
# 5	5580 #
# 6	7920 #

Note: The values given in Table A-1 may not exceed those given by Formula (A-2).

with a load factor of 1.7, and a strength reduction factor of 0.85 results in the following:

$$V = 0.5\ V_n$$

from which:

$$V = 0.3\ A_{vf} f_y \qquad \text{(A-1)}$$

where V is the force to be transferred (kips).

The shear transfer forces on a per bar basis are then computed and listed in Table A-1. The table values are based on $f_y = 60{,}000$ psi.

Transfer values for other bar sizes may be similarly computed, but the above sizes cover the more usual design situations.

Note: ACI 11.7. limits V_n to the lesser of the two following values:

$$0.2f'_c A_c \quad \text{and} \quad 800A_c$$

Using a concrete strength of 3,000 psi, the limiting value becomes:

$$V_n = 0.2 \times 3000 \times 12 \times \mathrm{t} = 7200\ \mathrm{t}$$
$$= 800 \times 12 \times \mathrm{t} \qquad = 9600\ \mathrm{t}$$

Where:

t is the distance across the shear plane, inches, and
V_n is computed for one linear foot along the length of the shear plane

Using this limiting value of V_n, and converting to transfer force, since

$$V = 0.5 V_n$$

then

$$V = 3600t \tag{A-2}$$

When this requirement is compared to the values given in Table A-1, it can be seen that the table values will govern the design for all reasonable widths of shear plane and bar spacing. Prudent engineering, however, requires verification, especially in those cases with shear planes of little width or with large, closely spaced reinforcing bars.

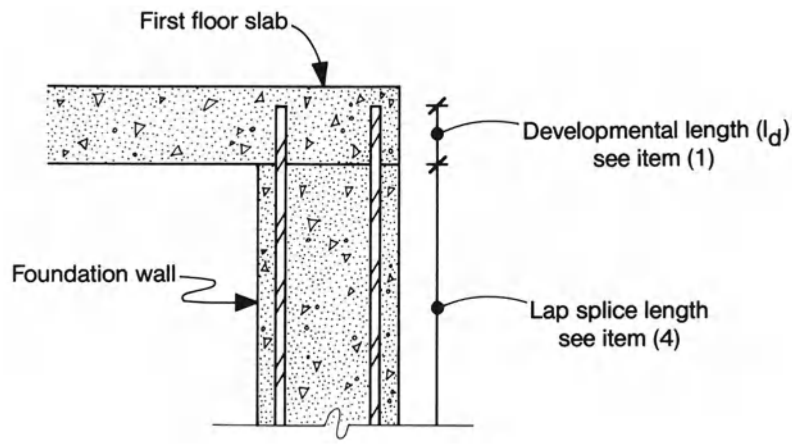


FIGURE A-2. Shear-friction at juncture of wall and slab.

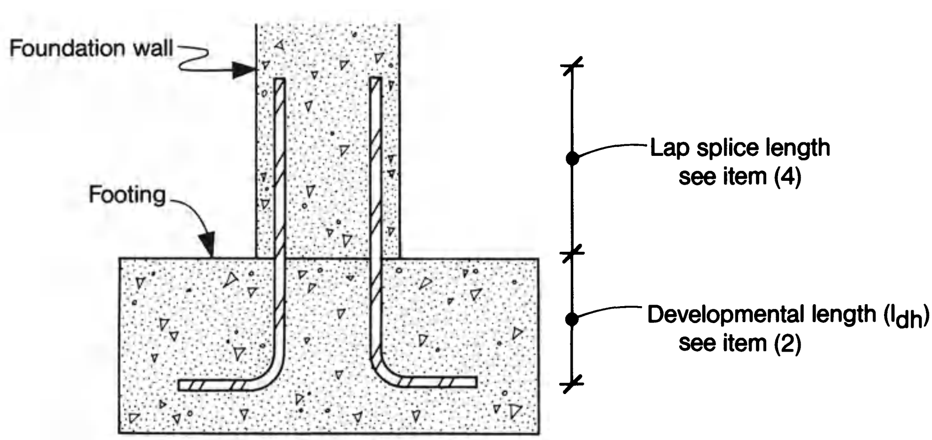


FIGURE A-3. Shear-friction at juncture of footing and wall.

A-2. SHEAR-FRICTION REINFORCING DETAILS

As noted previously, shear-friction may occur where lateral forces must be transferred from basement walls into the first floor slab. A typical detail of this condition is illustrated in Figure A-2.

Shear-friction may also occur where the wall meets the footing, as illustrated in Figure A-3.

A-3. DEVELOPMENT OF REINFORCING

The main difficulty with achieving a workable design based on the method of shear-friction is that of providing adequate development length for the reinforcing steel on both sides of the joint. Development length within the wall itself is no problem—the length used here will be that of the tension lap splice given in item (4) of Table E-1. There may be, however, a problem in providing the required development length in the footing, as given in item (2) of Table E-1, and there will almost certainly be a problem in providing the required development length in the first floor slab, as given in item (1) of the same table. These two items are repeated here in Table A-2, for the convenience of the reader. The numbers in parentheses reflect the reduced lengths which result from the use of the 0.80 and 0.70 modification factors itemized in Appendix E. The conditions from which these reductions result can be expected to occur in most foundation wall situations. It is left to the designer to determine whether these reductions are appropriate for any particular situation.

TABLE A-2. Development Length Requirements for Shear-Friction.

Bar Size	Straight Bars l_d, item (1)	Hooked Bars l_{dh}, item (2)
# 3	12″ (12″)	9″ (6″)
# 4	12″ (12″)	11″ (8″)
# 5	15″ (12″)	14″ (10″)
# 6	19″ (16″)	17″ (12″)

Note: For clarification of lengths given in the table, refer to Figures A-2 and A-3.

A-4. 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 then 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.

In the case of shear transfer across the cold joint of two elements cast at different times (similar to those indicated in Figures A-2 and A-3) the concept of shear-friction assumes that:

1. There will be slippage across the joint.
2. There will be sufficient roughness and lifting capability on the adjoining surfaces so that they will be forced apart.
3. This will cause the reinforcing steel to be stretched, thereby inducing tensile stress, which acts as a clamping force.
4. Hence 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 earth pressure be accomplished solely by the use of shear keys, as described in Appendix B.

APPENDIX B

Earth Pressure Transfer at Cold Joint by Shear Key

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-2, 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 straightforward.
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.

It should be noted that the side walls of the shear key between the footing and the wall are sloped. Refer to Section 9-7 for an explanation.

B-2. TYPICAL LOAD REQUIREMENTS

In order to have some quantitative idea of the earth pressures that must be transferred, consider the following:

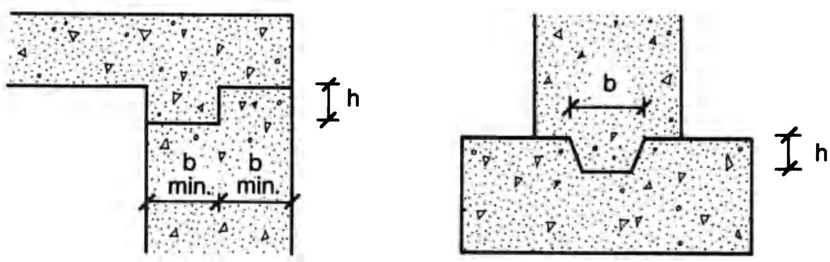


FIGURE B-1. Typical concrete shear keys.

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 pressure diagram see Figure 7-6.)
2. If the earth in the preceding example becomes completely saturated, the transfer forces increase to 2100 and 4200 pounds per linear foot, respectively. (See Figure 7-8 for pressure diagram.)

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

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.

The shear key must resist stresses induced by bearing, flexure, and shear, all as illustrated in Figure B-2.

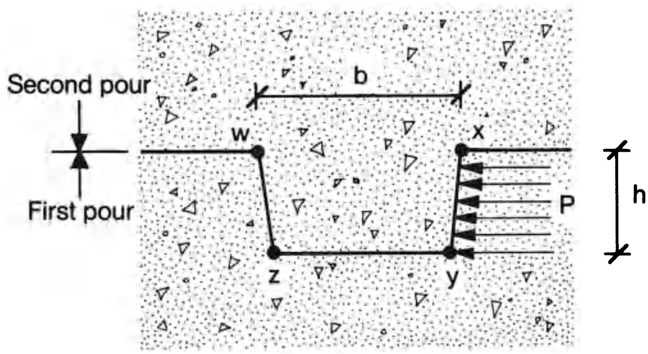


FIGURE B-2. Stress distribution in a shear key.

The stresses induced into the shear key by the external load P, and identification of the plane which resists them, are as follows:

1. Bearing on the vertical surface of contact, indicated by the line xy. The area which resists this bearing is:

$$A = 12h$$

2. Flexure on the plane of bending, indicated by the line wx. The section modulus which resists this flexure is:

$$I/c = 12b^2/6$$

3. Shear on the plane of shear, indicated by the line wx. The area which resists this shear is:

$$A = 12b$$

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 which follows.

Paragraph B.3.1(c) of Appendix B of ACI 318-83 [Ref. 5] specifies the following permissible bearing stress on the surface of contact:

$$f_{brg} = 0.3f'_c, \quad \text{psi} \tag{B-1}$$

Paragraph 9.5.2.3 of ACI 318-83 [Ref. 5] 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:

$$f_r = 7.5\sqrt{f'_c}, \quad \text{psi}$$

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:

$$f_b = 3.0\sqrt{f'_c}, \quad \text{psi} \tag{B-2}$$

This stress agrees with that presented in paragraph 18.4.1(b) of the same ACI publication.

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:

$$v_c = 1.1\sqrt{f'_c}, \quad \text{psi} \tag{B-3}$$

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 Calculations.

Type of Stress	General Formula	Allowable Stress[a]
Bearing f_{brg}	$0.3 f'_c$	900 psi
Flexure f_b	$3.0\sqrt{f'_c}$	164 psi
Shear v_c	$1.1\sqrt{f'_c}$	60 psi

[a] Stresses are based on 3,000 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	$f=\frac{P}{A}$	$900=\frac{P}{12h}$	$10{,}800h$
Flexure	$f=\frac{M}{S}$	$164=\frac{Ph/2}{12b^2/6}$	$\frac{656b^2}{h}$
Shear	$f=\frac{P}{A}$	$60=\frac{P}{12b}$	$720b$

[a] Transfer force has been computed in terms of pounds per linear foot of wall.

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:

$$15.0h > b > 1.1h$$

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.

In order to provide for a somewhat better proportioning of the shear key, and yet to have the design remain solely based on shear, the following relationship between b and h is recommended:

$$2.5h > b > 1.5h \qquad \text{(B-4)}$$

This limitation on the relative dimensions of width and height is both reasonable and practical, and should be used in all details relating to shear transfer at the top and bottom of the basement wall.

B-5. RECOMMENDED TRANSFER FORCE

The author has observed the spalling 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.

Shear keys should be designed to satisfy the two following formulas. The first provides for the recommended proportioning of height to width, as given by Formula (B-4), the second provides for the requirements of stress due to shear, as given in Table B-2:

1. $2.5h > b > 1.5h$
2. $P = 720(b - 1)$

TABLE B-3. Allowable Transfer Force on Concrete Sear Keys.

Width b	Height h	Transfer Force p[a]
4″	2″	2160 #
6″	3″	3600 #
8″	4″	5040 #
10″	5″	6480 #
12″	6″	7920 #

[a] Transfer force has been computed in terms of pounds per linear foot of key, with shear design governing, as per formula (B-4).

These formulas have been used to determine the allowable transfer force P for representative shear keys, as shown in Table B-3.

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 proportional 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 circular load q, in psf or ksf. Circular loads generally are 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.

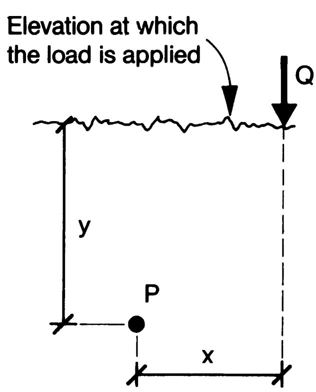
The procedures which follow provide for the determination of the vertical pressure induced at any point within the soil mass.

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 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,



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.

Where:

Q is the concentrated load, pounds
p_v is the vertical pressure induced at point P, psf
C_1 is an influence coefficient, values as given in Table C-1
x and y are distances as shown, feet

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 is illustrated in Figure C-2.

The intensity of this pressure can be computed with the following formula:

$$p_v = C_2 q$$

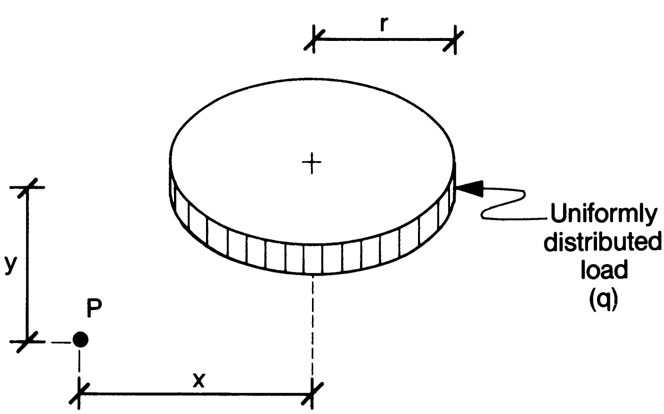


FIGURE C-2. Pressure induced by a circular load.

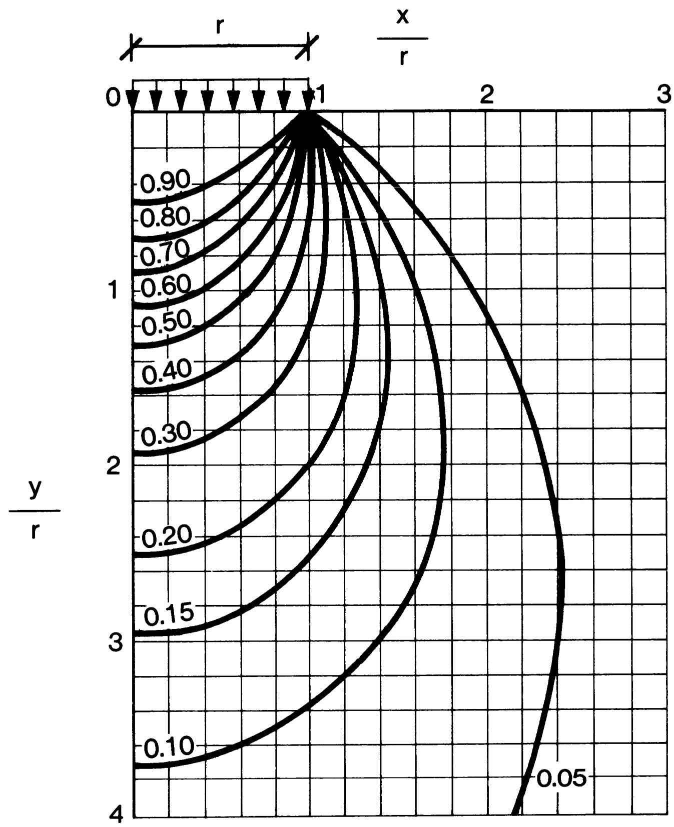


FIGURE C-3. Plot of the influence coefficient C_2. [Ref. 11]

Where:

q is the uniformly distributed circular load, psf
p_v is the vertical pressure induced at point P, psf
C_2 is the influence coefficient, values for which are given in Figure C-3

It should be noted that although theoretically not correct, this method can usually be applied to square areas having a side equal to twice the radius 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 the loaded area. This procedure is applicable for use with square, rectangular or continuous footings. The parameters required are given in Figure C-4.

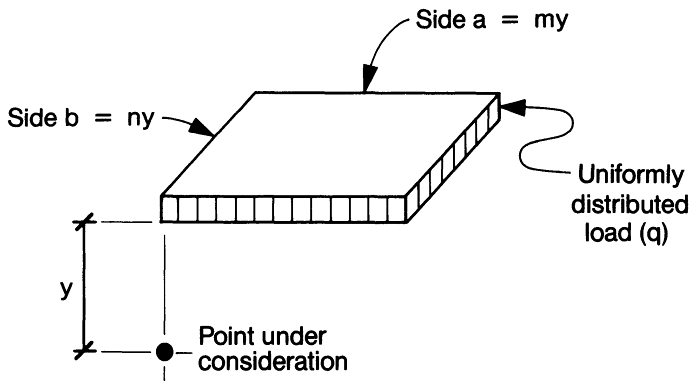


FIGURE C-4. Pressure induced by a rectangular load.

The intensity of pressure induced at point P is found as follows:

$$p_v = C_3 q$$

Where:

q is the uniformly distributed load, psf
p_v is the intensity of vertical pressure, psf
C_3 is an influence coefficient, given in Figure C-5

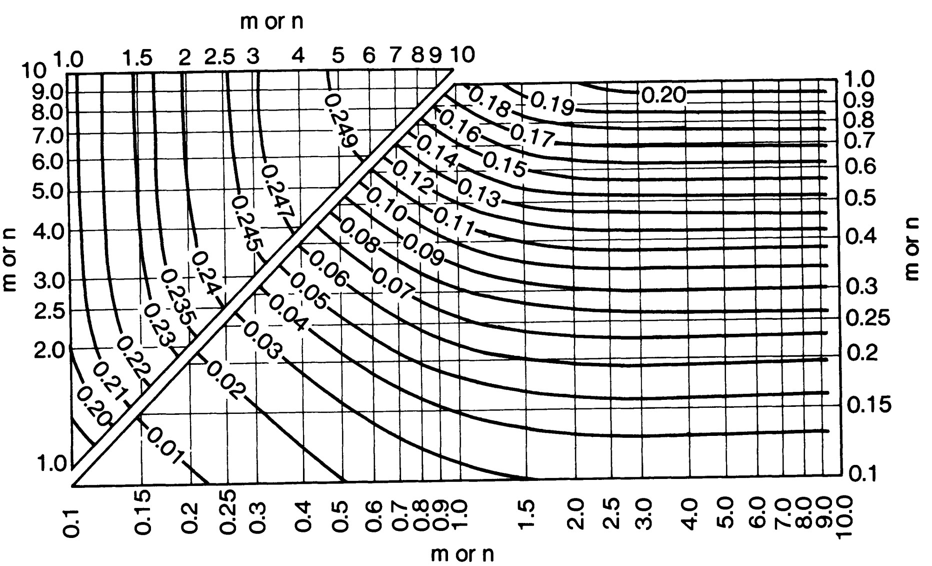


FIGURE C-5. Plot of the influence coefficient C_3. [Ref. 19]

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-6 and C-7, in which:

$$p_v = C_3 q$$

In Figure C-6 the pressure at point P is equal to the additive effect of the following areas:

$$AaPd + aBbP + bCcP + dPcD$$

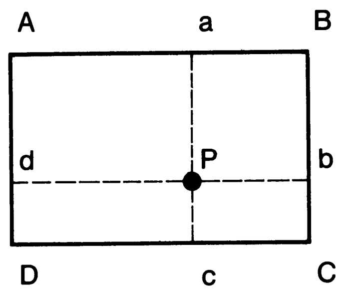


FIGURE C-6. Pressure induced by a rectangular load, point inside.

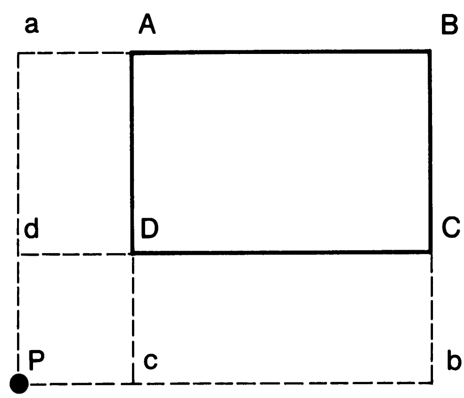


FIGURE C-7. Pressure induced by a rectangular load, point outside.

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.

In Figure C-7 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 is:

$$p_v = 0.273 \frac{400{,}000}{20^2} = 273 \text{ psf}$$

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 20 feet below it. The circular area has a radius of 10 feet.

Refer to Figure C-2. Calculate

$$\frac{x}{r} = \frac{8}{10} = 0.8$$

and

$$\frac{y}{r} = \frac{22}{10} = 2.2$$

From Figure C-3, the coefficient $C_2 = 0.20$. Therefore, the vertical pressure is:

$$p_v = 0.20 \times 6{,}000 = 1{,}200 \text{ psf}$$

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-4. 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 is:

$$p_v = 0.165 \times 8{,}000 = 1{,}320 \text{ psf}$$

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-6 for the method by which the large area must be divided into four smaller areas. Compute values for m and n in accordance with Figure C-4, then obtain the numerical value of the coefficient C_3 from Figure C-5. Compute p_v and record 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

Basement Slab on Ground—Empirical Design

D-1. GENERAL DETAILS

Basement slabs are usually cast on the ground. The only exception to this occurs when the soil beneath the slab cannot provide adequate bearing for the superimposed loads to which it will be subjected, in which case the slab must be designed as a suspended slab, probably using earth forms. It is the opinion of the writer that the soil beneath the slab on ground should have a minimum allowable bearing pressure of at least 1 tsf in order to be considered adequate for the purpose intended. It is further recommended that expansive soils having a plasticity index greater than 10 should generally be excluded from consideration as subgrade material. Refer to Article 11-9.

Prior to the installation of the stone base the subgrade should be leveled. Soft spots should be removed and filled. Hard spots should be cut out for a depth of at least twelve inches and filled. Borrow fill, if required, should conform to the requirements of Article 10-2.

The construction of a typical slab on ground is illustrated in Figure D-1.

Slabs on ground must be sized and reinforced to adequately serve the loads to which they will be subjected. Some slabs are lightly loaded, as would be the case in residences and most retail stores. 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 is the responsibility of the designer.

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

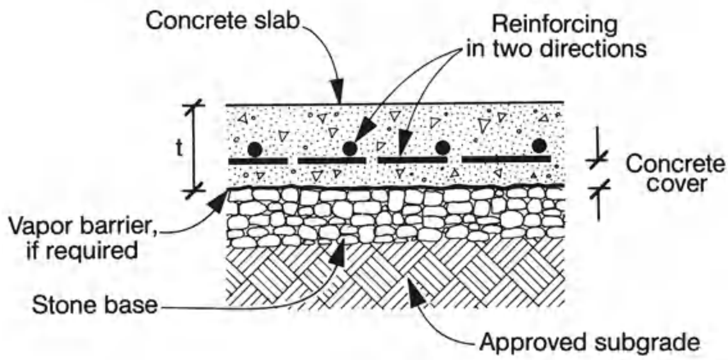


FIGURE D-1. Typical detail of a slab on ground. Concrete cover over reinforcing to be 1½″ for slabs up to 5″ thick, 2″ for slabs over.

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.

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.

Slabs supporting concentrated loads may be designed as inverted flat slabs. The concentrated loads are then thought of as being reactions to a uniformly distributed load caused by the subgrade acting upward against the underside of the slab.

For detailed information regarding the design and construction of slab on ground, the American Concrete Institute has several in-depth publications on this subject.

TABLE D-1. Empirical Design for Lightly Loaded Slabs on Ground.

Minimum Thickness	Minimum Ultimate Strength, psi	Reinforcing Each Way[a]	Maximum Load, psf
4″	2,500	# 3 @ 12″	100
5″	2,500	# 4 @ 18″	200
6″	3,000	# 5 @ 18″	400
8″	3,000	# 6 @ 18″	800

[a] Refer to Figure D-1 for location.

D-2. 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 90°, 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 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-2.

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. The laps given in that table are based on the requirements of Article A-3 of Appendix A, and the further requirements of ACI 318-83, paragraph 12.15.

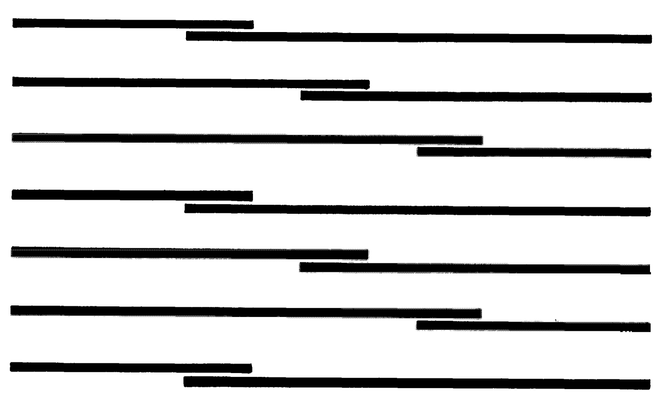


FIGURE D-2. Recommended arrangement for the splicing of reinforcing in slabs on ground.

TABLE D-2. Minimum Lap Length for Spliced Bars in Slabs on Ground.

Bar Size	Lap Length
#3	12″
#4	15″
#5	18″
#6	21″

Wire Mesh Alternate

Welded wire fabric, ASTM A185 and ASTM A497 (sometimes referred to as *wire mesh*) could be used as reinforcement in the more lightly reinforced slabs. It has been the author's experience, however, that except for 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 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-3. STONE BASE

Material

After it has been determined that the soil beneath the slab will perform satisfactorily as subgrade, a stone base is then installed.

Many contractors prefer to install the stone base, at least in part, before proceeding 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 conpaction 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

FIGURE D-3. Access as required for the delivery of stone base for use under slab on ground. [Ref. 7]

additional material is required, the contractor will leave an access through which the base material can be brought. Such as access is illustrated in Figure D-3.

Broken stone (crushed rock) well graded from 1½ inches to pea gravel is recommended as the material to be used for the stone base. Other materials, such as GW or SW or a GW-SW mixture are equally acceptable. It will generally be found, however, that the broken stone base will be more economical.

One of the functions of the stone base is to provide a place for the collection and disposal of any water or water vapor that might infiltrate the area. It is 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.

A 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.

The recommended thickness of the stone base is as follows:

1. Four inches for slabs up to and including six inches in thickness
2. Six inches for slabs over six inches in thickness

Compaction

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 10-4 of Chapter 10. Smaller areas should be compacted by the use of smaller, hand guided equipment as noted in Article 10-5 of the same chapter.

Compaction should result in a well-seated, well-graded stone base, similar to that illustrated in Figure 10-5(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.

D-4. GROUND WATER

All of the foregoing paragraphs assume that the occurrence of ground water need not be considered. When ground water does occur then the slab, and all details of waterproofing and water control, must be designed by engineers knowledgeable in the problems relating to hydrostatic pressure.

APPENDIX E

Dowels for Load Transfer into Footings

E-1. GENERAL CONSIDERATIONS

Purpose of Dowel

All walls and columns contain vertical reinforcing bars, which are designed to carry their proportional share of the total load carried by the element. This load may be tension or compression. As discussed in Article 5-6, 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 5-5 of Chapter 5.

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 common length of embedment 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 over the length of embedment of the reinforcement.

Typical Dowel Requirements

The vertical length of each dowel is the sum of two individual lengths:

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.

A typical detail indicating the development and lap splice lengths of a dowel is illustrated in Figure E-1, in which the radius about which the bar is bent shall be $3d_b$ for #3 to #8 bars, and $4d_b$ for #9, 10, and 11 bars.

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 use of dowels, for the purpose of transferring vertical load from wall or column reinforcing into the footing
2. Concrete to have a unit weight of 150 pcf, and an ultimate compression strength of 3000 psi
3. Reinforcing steel to have a yield stress of 60000 psi

For additional requirements in the use of these and other materials, refer to ACI 318-83 [Ref. 5].

(1) Development length l_d for straight bars—tension—ACI 12.2:

$$l_d \text{ shall equal } l_{db}\text{, but not less than } 12''$$

The 12″ restriction is waived in the computation of lap splice lengths, as given in item (4).

For #11 bars and smaller:

$$l_{db} = 0.04\, A_b f_y / \sqrt{f'_c}, \quad \text{but not less than } 0.0004 d_b f_y$$

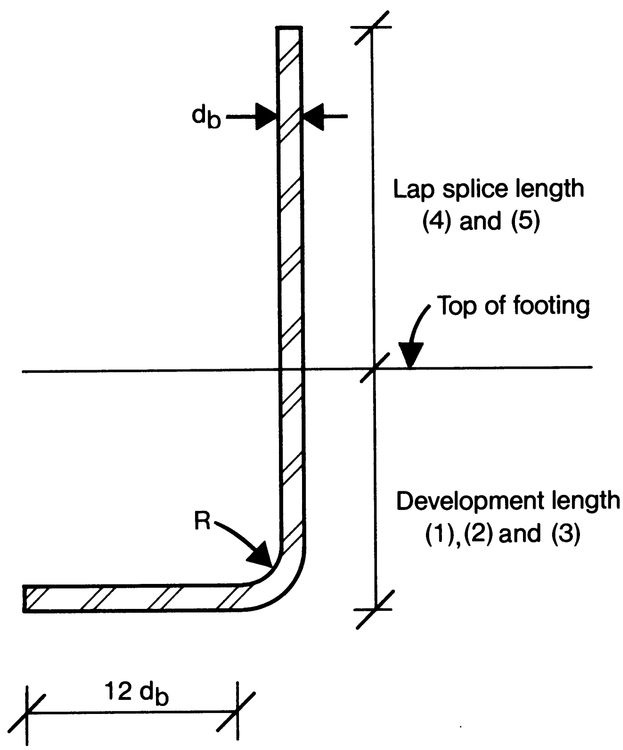


FIGURE E-1. Typical dowel detail.

(2) Development length l_{dh} for hooked bars—tension—ACI 12.5:

l_{dh} shall equal l_{hb}, but not less than $8d_b$ nor 6″

For #11 bars and smaller:

$$l_{hb} = 1200d_b / \sqrt{f'_c}$$

(3) Development length l_d—compression—ACI 12.3:

l_d shall equal l_{db}, but not less than 8″

For #11 bars and smaller:

$$l_{db} = 0.02d_b f_y / \sqrt{f'_c}, \quad \text{but not less than } 0.0003d_b f_y$$

(4) Lap splice length—tension—ACI 12.15. Assuming a Class B splice, the lap splice length shall be:

$1.3l_d$ from (1), but not less than 12″

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	19	17	17	25	23
7	26	20	20	34	27
8	35	22	22	45	30
9	44	25	25	57	34
10	56	28	28	72	39
#11	68″	31″	31″	89″	43″

Note 1. Refer to Figure E-1 for location where these lengths are applicable.
Note 2. Table values are applicable only for 3,000 psi normal weight concrete and 60,000 psi yield stress steel.

A Class B splice is defined as one in which no more than 50% of the bars are spliced within the required lap length.

(5) Lap splice length—compression—ACI 12.16. The lap splice length shall be l_d from (3), but:

$$\text{not less than } 0.0005 d_b f_y \text{ nor } 12''$$

Lengths based upon the preceding requirements, and as applicable to wall and column dowels in footings, are given in Table E-1. Table values for items (1), (2) and (4) may be modified as follows:

1. Use 0.80 times the l_{db} lengths given in (1) and (4) when the dowels are spaced laterally at least 6″ on center, with at least 3″ clear from the face of the member to the edge of the bar, measured in the direction of the spacing. In no case, however, shall the resultant length be less than 12″.
2. Use 0.70 times the l_{hb} lengths given in (2) when the side cover (normal to the plane of the hook) is not less than $2\frac{1}{2}''$.

For materials and/or situations of use not covered by Table E-1, refer to Chapter 12 of ACI 318-83.

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) in Table E-1. Without hooks, the footing may have to be increased to an unreasonable thickness just to accommodate the longer development length. In footings, therefore, hooks are almost universally used for tensile reinforcing in preference to straight bars.

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 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) of Table E-1. 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 shown 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 follows: *Buoyancy is the term used to indicate 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 displaced 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 worst anticipated condition, which will be at high water table.

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 filled with oil having a density of 50 pcf. The total weights are as follows: tank = 3,600 #, oil = 30,200 # and concrete pad = 23,600 #.

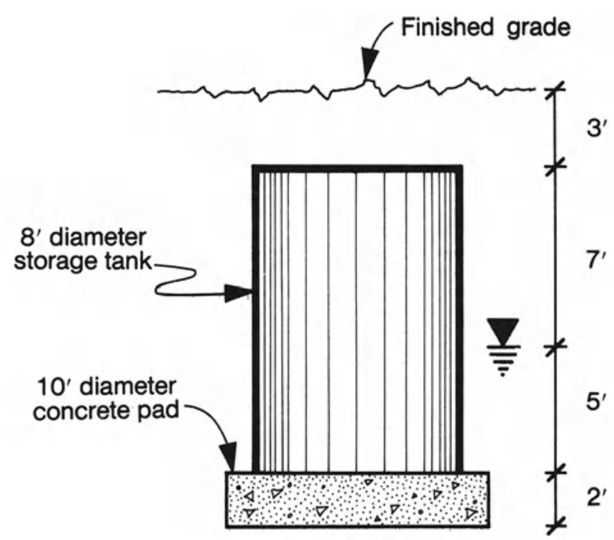


FIGURE F-1.

Volume of water displaced by partially submerged tank:

$$V = \frac{\pi\ 8.0^2}{4} \times 5.0 + \frac{\pi\ 10.0^2}{4} \times 2.0 = 408.4\ \text{CF}$$

$$\text{Bouyancy W} = 408.4 \times 62.4 = 25{,}500\ \#$$

With tank full of oil:

$$\text{Total weight} = 3{,}600 + 30{,}200 + 23{,}600 = 57{,}400\ \#$$

$$\text{Buoyancy} = 25{,}500\ \#$$

$$\text{Safety factor against uplift} = \frac{57400}{25500} = 2.25$$

Example F-2

Required: To determine the condition of buoyancy of the tank illustrated in Example F-1, assuming the tank to be empty.

$$\text{Total weight} = 3{,}600 + 23{,}600 = 27{,}200\ \#$$

$$\text{Buoyancy} = 25{,}500\ \#$$

The tank is very close to lift-off.

Example F-3

Required: To suggest possible solutions for the condition of flotation as described in Example F-2.
Solution #1: Anchor tank to concrete pad with the proper number of cast-in-place bolts, and anchor pad to sound bearing with post-tensioned, grouted tendons.
Solution #2: Anchor tank as before. Pour a thicker pad.

As an example of the second method, assume a 5 foot thick pad. With 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 partially submerged tank:

$$V = \frac{\pi\ 8.0^2}{4} \times 5.0 + \frac{\pi\ 10.0^2}{4} \times 5.0 = 644\ \text{CF}$$

$$\text{Buoyancy} = 644 \times 62.4 = 40{,}200$$

$$\text{Safety factor against uplift} = \frac{62500}{40200} = 1.55 > 1.50 \quad \text{Satisfactory}$$

Example F-4

Required: To determine the effect of ground water on the building indicated in Figure F-2.

A large scale detail of the construction of the basement floor is shown in Figure F-3. On a square foot of floor area:

$$\text{Weight of basement slab} = 1.67 \times 150 = 250 \text{ psf}$$

$$\text{Buoyancy} = 7.67 \times 62.4 = \underline{479 \text{ psf}}$$

$$\text{Net uplift} = 229 \text{ psf}$$

On a per column basis:

$$\text{Net uplift} = 0.229 \times 30^2 = 206 \text{ kips}$$

For a safety factor of 2, the column load due to dead load only should be

$$2 \times 206 = 412 \text{ kips.}$$

Note: When considering uplift, only those forces known to be continuously present are counted upon to resist the force of buoyancy.

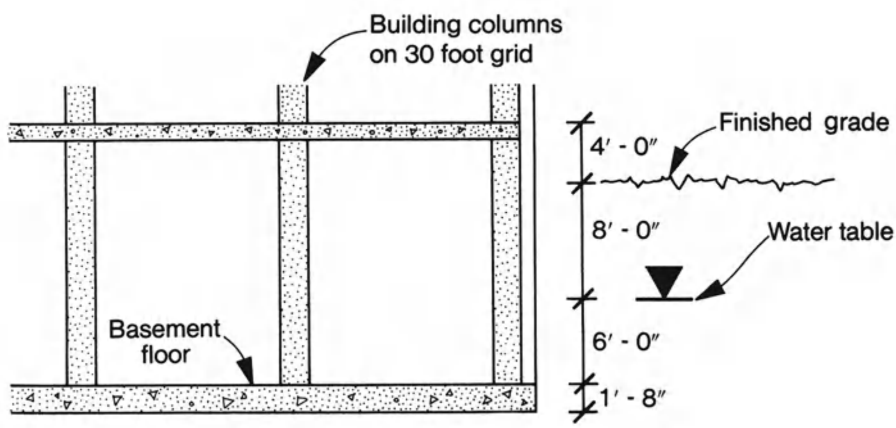


FIGURE F-2.

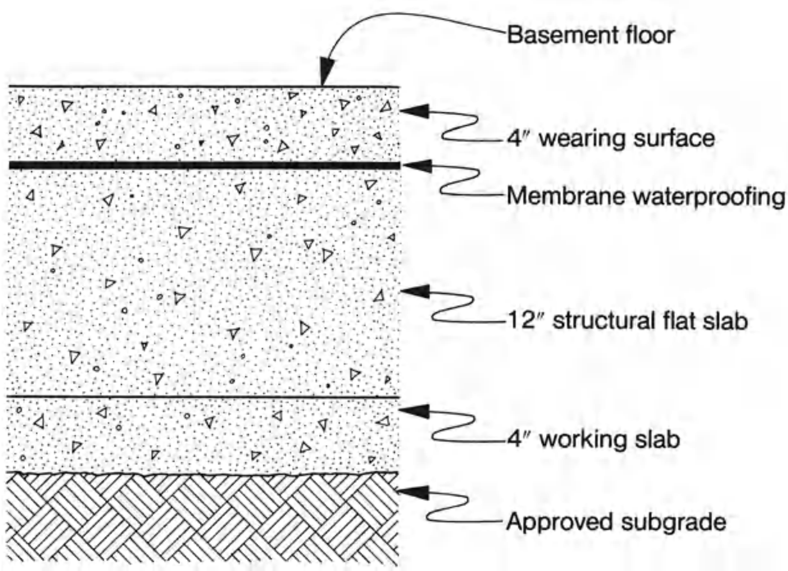


FIGURE F-3.

Example F-5

Required: Assume that the ballast provided by the column in Example F-4 is only 232 kips. The 180 kips deficiency is to be made up by increasing the thickness of the basement floor construction. What additional thickness is required?

$$(150 - 62.4)\, t = \frac{180{,}000}{30 \times 30}$$

From which

$$t = 2.28 \text{ feet}$$

$$\text{Total floor thickness} = 1.67 + 2.28 = 3.95$$

Use 48″.

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