Design of Shallow Foundations

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Abstract: This book delivers a balanced presentation of both soils and structures as they relate to shallow foundations. From soils, it includes a treatment of relevant soil properties and soil mechanics at shallow depths. From structures, it includes a summary of loads on foundations and the deformations produced by such loads. The focus, however, remains at the founding line where the particular structural design is matched to the particular soil conditions. This book is intended for use by students as well as the practicing engineer.

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PREFACE

The study of foundations is invariably treated as two separate and distinct topics: the study of shallow foundations and the study of deep foundations. The concepts, approaches and practices involved in these two topics are quite different; it is a very natural choice to study them separately. This book deals with the first topic, the study of shallow foundations. Traditionally, the design of routine shallow foundations is presented at the undergraduate level, while the study of deep foundations is ordinarily presented in more advanced work, quite often at the graduate level.

The design of shallow foundations necessarily involves two disciplines: soil mechanics and structural mechanics. Specialists in soils are somewhat inclined to view foundation design as a soils problem with implications in structures. Specialists in structures are similarly inclined to view foundation design as a structures problem with implications in soils. In this text, only the most common case is considered, where foundation design is an interwoven indistinct melding of the two disciplines.

It is intended that this book will set forth a balanced presentation of both soils and structures as they relate to foundations. From soils, it includes a treatment of relevant soil properties and soil mechanics at shallow depths. From structures, it includes a summary of loads on foundations and the deformations produced by such loads. The focus, however, remains at the founding line, where the particular structural design is matched to the particular soil conditions.

The approach used in this book has been simplified where such simplification does not diminish accuracy. However, there should be no delusions about high levels of accuracy in foundation design, where at best, concrete properties may be accurate to two places and soil properties may be accurate to one. Elaborate methods of analysis requiring high levels of predictability in the material properties are avoided.

In any field of study as new as soil mechanics, numerous fragments of information will continually be generated as research expands piecemeal in all directions. It should be expected that some of these fragments will be shown in future years to be relevant, some merely peripheral. There has been a concerted effort in the preparation of this textbook to bypass such a mass of fragmented information and to focus on a single line of "established" and proven methods. Some of these established methods will undoubtedly be superceded in future years as newer, better and more efficient methods emerge. For now, the methods presented here have been chosen simply because they are known to work. They will not cease to work just because more accurate methods or more comprehensive methods or more efficient methods are developed in the future.

This book is intended for use both by students as well as persons already in practice. It is intended to include architecture, construction and engineering technology as well as civil engineering. It is presumed that the user will have taken the usual preparatory courses in statics and strength of materials. It is also presumed that the user will be fully familiar with concepts of stress and strain, to include the Mohr's circle analysis for state of stress at a point.

The Imperial (British) system of weights and measures is used exclusively in this text. However, in deference to the publisher's policy of including a ready and convenient means of conversion to SI units, a table of common conversion factors is included at the beginning of the text.

The exclusive use of Imperial units rather than SI units is a matter of practicality rather than preference. In Memphis, for example, there has been only one application in memory for a building permit in metric units. That application (1996) was from a Canadian engineering firm on behalf of their Canadian client. All materials manufactured to metric standards (doors, windows, plywood, plumbing fixtures, pipe, etc.) were presumably shipped from Canadian suppliers; they are not generally available in the U.S.

Since the conversion to SI units is not proceeding at a rapid pace, the short life of about eight years for a book such as this requires that currently familiar terms, phrases and measurements be used. In consideration of market size and market appeal in the U.S., the author has chosen to stay with the more familiar Imperial units.

In U.S. literature, the practice of providing parenthetic SI units following each use of Imperial units seems to promote clutter with no apparent promotion of conversion. Since there is a conscious effort in all of the author's books to reduce clutter, the practice of using parenthetic SI units is avoided here.

As with his earlier books, the author is again indebted to his wife Sherry, who typed the original manuscript of the text. Her unwavering support of these speculative ventures is gratefully acknowledged.

Samuel E. French, Ph.D., P.E.
Martin, Tennessee, 1998

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PART I

TYPES OF LOADS AND TYPES OF SOILS

Chapter 1

APPLICATIONS*

Shallow Foundations in Modern Construction

By far the most common structural foundation in today's construction industry is the shallow foundation. Other types of foundations such as piles, piers, caissons and similar deep foundations are used primarily for major structures, not for the ordinary smaller structures that constitute the overwhelming majority of all construction. This book is devoted to the study of shallow foundations for these smaller structures. Over the centuries, such foundations have been proven to be economical, serviceable and reliable.

As with any structural system, shallow foundation systems have limitations in their use. Limitations arise from the load-bearing capacity of soil, the magnitude of loads, the configuration of the structure, the type of materials, and any number of other conditions that can occur in a project. Many times, these limitations can be overcome, sometimes at little cost, sometimes at great cost. Wherever shallow foundations can be made to work, however, they will usually be more economical by a wide margin than the nearest alternative.

In succeeding chapters, the more common conditions that affect the use of shallow foundations will be introduced and analyzed. Because there are so many special circumstances that could occur, an introductory volume such as this is necessarily limited to the most common applications. These common types of foundations, structures and soils are summarized in this chapter.

Common Types and Configurations of Buildings

One of the more common structural systems used for small buildings is the braced frame. Two typical examples of a braced frame are shown in Fig. 1-1. The structural frame of Fig. 1-1a is usually built of steel. The shearwall bracing of Fig. 1-1b is usually built of concrete although masonry shearwalls are quite common.

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* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

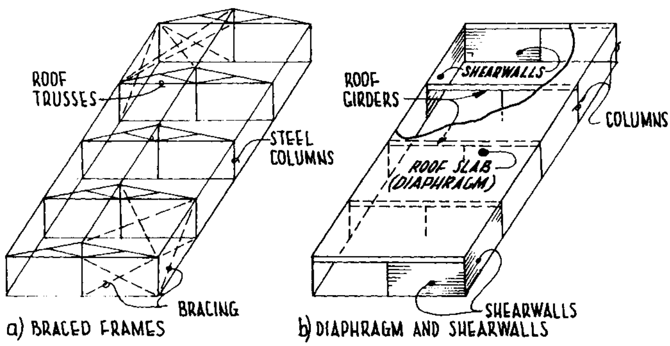


Figure 1-1 Typical Braced Frames

The distribution of loads in braced frame systems is readily determined using only simple statics. Gravity loads (live and dead) are carried by columns; lateral loads (wind and earthquake) are carried by a separate bracing system. The floors or roof are carried by framed girders or trusses. Columns carry primarily vertical loads, but some small moments can occur due to eccentricities of loading.

Lateral wind and earthquake loads acting against a braced frame is resisted only by those footings that directly support the lateral bracing system; no other footings in the structure resist lateral loads. In addition to producing a lateral force against a structure, wind and earthquake also produce an overturning moment on the structure as a whole. This overturning moment is resisted by a restoring moment (a vertical couple) at the footings of the bracing system.

Another very common type of structural system is the rigid frame, shown in Fig. 1-2. In this system, wind and earthquake forces are resisted entirely by bending of the columns; there are no braced panels or shearwalls. Due to such bending, the footings will be subject to rotations wherever the columns are rigidly fixed to their footings.

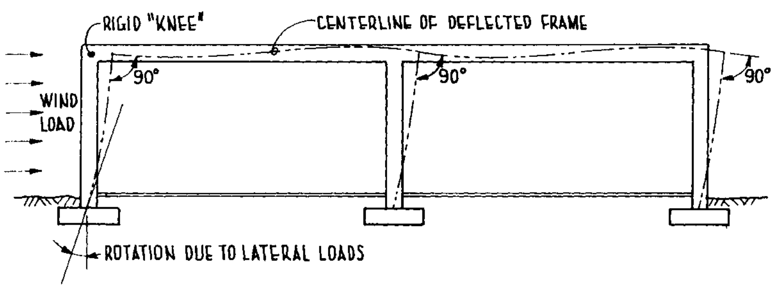


Figure 1-2 Typical Rigid Frame

Loads delivered to the soil by each column in a rigid frame will include vertical dead and live loads as well the lateral shearing force due to wind and earthquake. Also, an additional vertical force on all columns will be induced by the overturning effects of wind and earthquake. The foundation design must, of course, include all such effects.

In common applications of the two systems, braced frames are more likely to be used for low buildings and rigid frames for higher buildings. There are no sharp distinctions, however. One of the most common single-story structures in today's industry is the prefabricated steel building shown in Fig. 1-3, which is a rigid frame in one direction and a braced frame in the other.

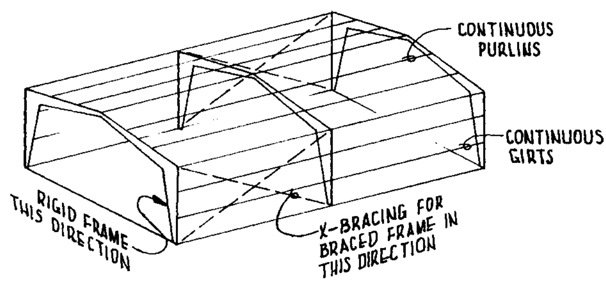


Figure 1-3 Typical Prefabricated Steel Building

Footing loads for these common types of structural systems can be carried by shallow foundations as long as the height of the building is not too great. For steel buildings, a height of some six to eight stories on shallow foundations is usually feasible. For concrete buildings, a limit of only four or five stories is common, due primarily to the heavier weight of concrete buildings.

The limitation on heights is simply one of economics. As height increases, column loads increase and footing sizes increase proportionately. When the total area of footings exceeds about one-third of the building footprint, other types of foundations could be more economical.

At the suggested limitation of six to eight stories, column loads in routine structures should not be more than about 180 tons, assuming a column module of some 20 to 30 feet. At a maximum soil pressure of about 4000 pounds per square foot, the corresponding footing size is then roughly 10 to 12 feet square. It may be concluded, therefore, that when footing sizes begin to exceed about 12 feet, foundations systems other than shallow footings might be worth considering.

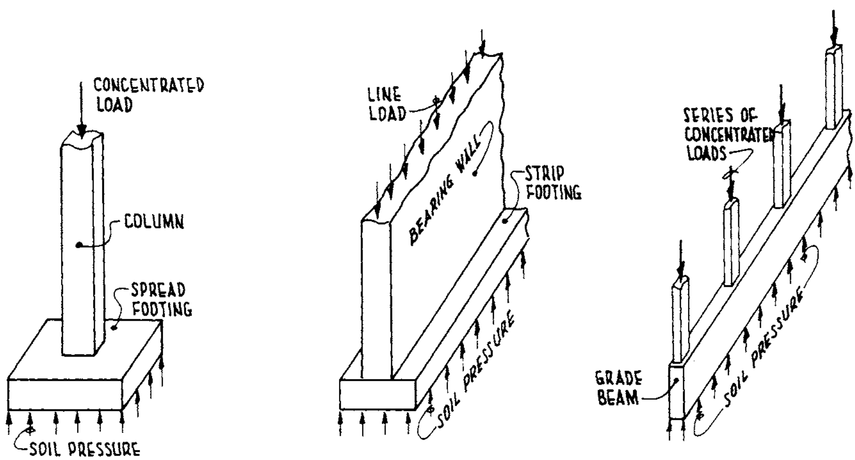
In many small buildings, a heightened first floor is a common architectural feature. In such designs, the columns at the first floor are quite long, up to 16 feet or even more. When the extra length could affect the footing design, that length is included in the analyses presented in subsequent chapters.

Walls or columns cantilevered out of a shallow footing are not considered in this textbook. The "flagpole" analysis for such cases is readily apparent but such cantilevered columns are not used in American practice.

Common Types and Configurations of Foundations

The most common types of shallow foundations fall into three descriptions:

1. *Spread footings* supporting a concentrated load, such as a load delivered by a column (Fig. 1-4a).
2. *Strip footings* supporting a line load, such as a load delivered by a bearing wall (Fig. 1-4b).
3. *Grade beams* supporting a repetitive series of concentrated loads, such as a load delivered by a line of several columns (Fig. 1-4c).



a) SPREAD FOOTING b) STRIP FOOTING c) GRADE BEAM

Figure 1-4 Typical Shallow Foundations

All three of the footings shown in Fig. 1-4 fulfill their function by distributing a concentration of load over a larger area. The footings are said to be *founded* on the soil at the *founding line*. The area at the bottom face of the footing that bears on the soil is descriptively called the *contact area*, or sometimes, *footprint*.

The isolated spread footing shown in Fig. 1-4a should always be centered under the column. Loads applied off center can produce wastefully high peaks in soil bearing pressure and are to be avoided. Where a line of two or more columns are placed on one footing, the required symmetry is obviously reduced to only one axis.

Strip footings distribute line loads outward as indicated in Fig. 1-4b. The need for symmetry is readily apparent. There is no stress in a strip footing in the long direction; the wall itself must be designed to sustain any variations in load that may occur along its length.

A grade beam is a continuous beam that is subject to flexure longitudinally, loaded by the line of columns it supports. It is supported at the contact area by the upward soil pressure. The loading system of the grade beam of Fig. 1-4c may look more familiar if the figure is viewed upside down.

The contact area of a shallow foundation is kept a specified distance below surface level. That distance is called *depth of founding* and is denoted D_f. Typical measurements of D_f are shown in Fig. 1-5.

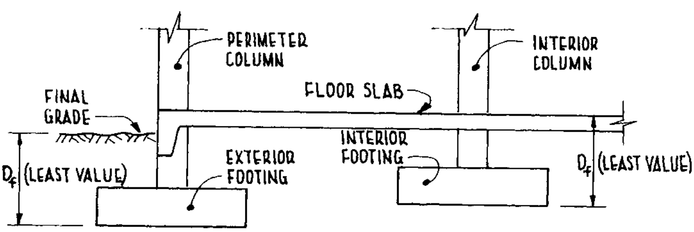


Figure 1-5 Measurement of Depth of Founding

The minimum depth of founding to be used in foundation design is usually prescribed by the building code. In temperate climates, the minimum depth is set to provide enough cover to prevent frost heave. Minimum depth of founding is rarely less than 16 inches and may be considerably more depending on local conditions.

The maximum depth of founding that might be used and still have the footing classified as a "shallow" foundation is quite arbitrary. Significant changes begin to occur in soils at depths exceeding about six to eight feet; such changes occur due both to the higher confining pressures and to the more stable moisture contents. And, for the sake of foundation construction, a practical limit for unbraced or open excavations is also about six to eight feet. Due to such considerations, a maximum depth of about eight feet is taken here as an arbitrary but reasonable limit on the depth of founding D_f.

The contact area of a shallow foundation is kept horizontal. When a bearing wall is to be located along a sloping grade, both wall and footing are stepped to follow the grade as indicated in Fig. 1-6. The wall and footing are stepped as needed to keep the depth of founding greater than the minimum depth of 16 inches required by code.

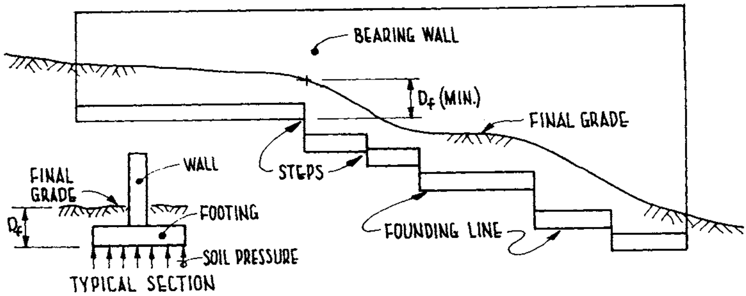


Figure 1-6 Stepped Footing along a Slope

Presumably, the area of a shallow foundation could be made any size required. As noted earlier, however, there are practical limits. When sizes become too small, stress concentrations can cause problems. When sizes become too large, simple economics suggests that other types of foundations might be considered. A minimum size of 2 feet and a maximum size of 12 feet are observed in this text.

Common Soil Pressures and Settlements

For the sake of comparison, soils having an allowable bearing pressure greater than about 6000 psf would be classified as one of the better foundation materials. It would be unusual for a soil having that much strength to pose serious problems with settlements. For lower bearing strengths, 5000 psf and less, both strength and settlements are increasingly likely to be of concern.

Unfortunately, it is the soils that have an allowable bearing capacity below about 5000 psf that are most likely to be encountered at shallow depths. At these shallow depths, the soils have probably been subject to weathering, water percolation, chemical action, desiccation and other disturbances that reduce strength. Even with capacities less than 5000 psf, however, it is usually more economical to utilize these shallow soils than to try to go deeper.

The lowest reasonable value that may be used for allowable soil pressure is not limited except by economics. At low pressures, footing sizes can simply be too large to be economically feasible. In general, a soil having an allowable pressure below about 1000 psf would not be used to support structures. For such soils, a mat or raft foundation might be considered. (A mat is a very large flexible "plate" covering the entire building footprint.) Alternatively, deep foundations (piles) may be appropriate.

As indicated in Fig. 1-7, pressure-settlement curves for most soils are are not usually linear (straight line) where pressures are so high that they approach ultimate strength. Back at service levels, however, small variations are seen to be very nearly linear. Within the lower range of pressures used in foundation design, pressure-settlement relations are assumed herein to be linear, that is, a 50% increase in service pressure will produce a 50% increase in deformation.

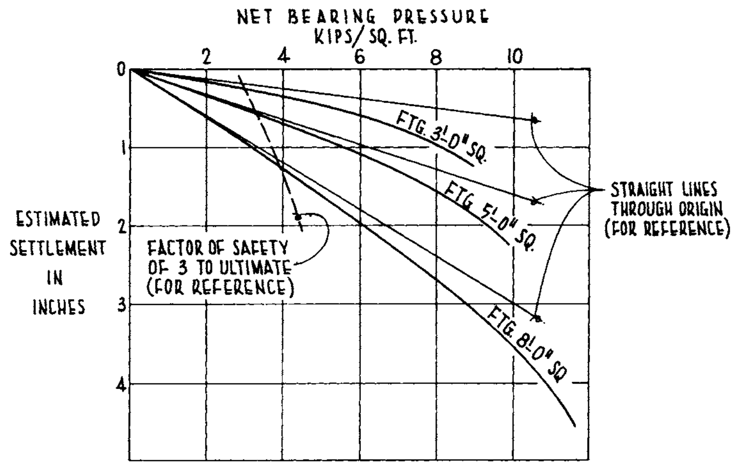


Figure 1-7 Typical Pressure-settlement Curves

The soils of interest here are ordinary soils (whatever that means) having an allowable bearing pressure within the foregoing limits of 1000 to 5000 psf. These soils will be assumed to have equally ordinary pressure-settlement characteristics. The response of such soils to load can be expected to be reasonably predictable.

As strength improves in these soils, rigidity also improves. Consequently, when a weaker soil is subjected to its low allowable bearing pressure and a stronger soil is subjected to its higher allowable bearing pressure, it can be expected that both will deform by roughly the same amount. As one may suppose, such things are borne in mind when the allowable bearing pressures are being established.

There are soils that respond to load quite differently than the soils just described. Means to identify these unconventional types of soil are discussed further in later chapters. Wherever such problem soils exist, it is usually common knowledge locally to those in the building industry; it is very easy to find out whether one should seek help. The design of foundations in a problem soil is best relegated to experienced professionals.

A common example of such a problem soil is a clay soil that has been heavily compressed at some point in its geologic past. Such a clay could be subject to erratic swelling when exposed to water. Such clays are called *active* or *expansive* clays and require special consideration when used as a foundation material. They are specifically excluded from the "conventional" soils treated in this book, although they are discussed further in Chapter 11.

Standard Test Specifications

It is a feature of an engineering soils investigation that certain tests will be run on the soil to determine its engineering properties. Over the years, such tests have become standardized. Specifications are now available that prescribe exactly the procedures, equipment, quantities and timing (if appropriate) to be used in conducting these tests.

The more common standard soil tests that are referenced in this textbook are contained in the specifications of the American Society for Testing and Materials (ASTM), Volume 19. For identification of a specific test, the ASTM designation for the test is a number, such as ASTM D698-89, in which the number following the dash indicates the year of the latest review or revision. When an ASTM test designation appears in this book without such a dash number, it should be understood that the most recent revision is intended.

A second set of soil test specifications in common use is that of the American Association of State Highway and Transportation Officials (AASHTO). The designation of a specific AASHTO test is also given by number, followed by the year of the latest revision, such as AASHTO T-180-74. In addition to test specifications for soils, AASHTO also publishes extensive sets of design specifications, construction specifications and recommended practices that have also become standards in the industry.

For concrete, the specifications of the American Concrete Institute (ACI) are used exclusively in this book. A specification issued by ACI will bear the number of the committee that is responsible for originating and maintaining the specification, as well as a number indicating the year of the latest revisions. For example, ACI 318-89 is the specification "Building Code Requirements for Reinforced Concrete" as reported by ACI committee 318 and issued by ACI in 1989. Lesser-known sources of tests and specifications in the text are identified as they are encountered.

Useful Approximations

It will be noted repeatedly in subsequent discussions that the engineering properties of soil is subject to considerable variation. Even when determined very accurately in a test lab, the properties of the soil can vary significantly from point to point throughout the field stratum. As a consequence, calculations involving soil properties should not be expected to have an overall accuracy better than $\pm 50\%$.

Under such circumstances, reasonable approximations may be used without disturbing the overall accuracy. A typical example of such an approximation is the uncracked rectangular moment of inertia $I = bh^3/12$ for concrete columns even when the concrete section may be cracked, where I is the moment of inertia, b is the width of the section in bending and h is the height of the section in bending. Similarly, the section modulus in bending may be taken as that for an uncracked rectangular section, $S = bh^2/6$.

It is acknowledged that the unit weight of soil, γ, and the unit weight of concrete, γ_c, are sizeably different: for soil, γ = 90 to 125 pcf, and for concrete, γ_c = 145 pcf. In soil mechanics, the disparity is usually ignored; the weight of a concrete footing is commonly assumed to be equal to the weight of the soil it displaces.

Chapter 2

GRAVITY LOADS ON FOUNDATIONS*

General Categories of Loads on Structures

Loads on structures are generally broken into two broad categories, gravity loads (dead and live) and lateral loads (wind and earthquake). All loads are treated as static loads. In truth, live loads, wind loads and earthquake loads may be highly dynamic, but in practice such loads are converted into equivalent static loads and applied as static loads rather than as dynamic loads.

Environmental loads are a special type of loading that may occur on structures. Typical of such loads are snow, ice, sand accumulation, ponding rain and other such regional environmental hazards. The proper application of these loads is highly dependent on local practices; no attempt is made here to account for a multiplicity of such special loadings.

In later chapters, an accurate analytical method is developed for selecting the size and shape of the contact area of a footing. In the methods presented there, only the vertical design load is used (no column moments are needed). These vertical design loads may be determined using the simplified techniques and methods developed in this chapter; more sophisticated methods are not necessary.

Allowable Footing Pressures for Gravity Loads

In the design of footings to support vertical gravity loads, two conditions must be met:

1. The allowable strength of the soil cannot be exceeded.
2. The allowable settlement of the footing cannot be exceeded.

In order to satisfy these two conditions, two allowable bearing pressures are established. Means to establish these pressures are presented in Chapter 6 and Chapter 8.

* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

The two allowable pressures for gravity loads are:

- An allowable bearing pressure p_a is determined to satisfy strength limitations, with no consideration given to the corresponding settlement limitations
- An allowable bearing pressure p_a'' is determined to satisfy the settlement limitations, with no consideration given to the corresponding strength limitations

These two pressures are used with the appropriate loads to determine the required footing size.

To satisfy the limitations in the strength of the soil, the allowable pressure p_a for strength may not be exceeded whenever maximum gravity loads occur. Maximum gravity loads consist of the combined maximum dead and live loads that can occur at any one time, denoted DL + LL.

It should be recognized that maximum live load will likely be present only a few times in the service life of the structure. Even so, the soil must safely support this maximum load regardless how infrequently it may occur.

To satisfy the limitations on settlement of the footing, the allowable bearing pressure p_a'' may not be exceeded under the maximum sustained gravity loads. It is a characteristic of soils, however, that settlements occur slowly, over a period of weeks or months. Consequently, the only portion of the load that will contribute to settlement is the portion that remains in place for a sustained length of time.

It is common practice to assume that only about half of the live load is in place over a sustained period of time. The sustained load is therefore commonly taken to be the dead load plus about half the live load, denoted DL + 50%LL.

Gravity Loads

As implied by the name, the gravity loads acting on a building are those due to the attraction of gravity on the fixed masses of the building itself (dead loads) and those due to the attraction of gravity on transient masses (live loads). Gravity loads usually act vertically, but in some cases they may act horizontally (e.g., lateral soil loads against a basement wall). Gravity loads are of much longer duration than lateral loads (wind and earthquake), which, by comparison, are of extremely short duration.

Within the general classification of gravity loads, the distinction between dead loads and live loads is an important one. Since dead loads can be very accurately determined and transient live loads can only be estimated, structures are generally designed for a lower factor of safety for dead loads than for live loads. Further, earthquake forces will be determined later by computing the lateral accelerations of dead loads; live loads are excluded. And still further, settlements of foundations will be based on long-term loads, requiring a distinction to be drawn between long-term dead loads and all other loads. Reasonable care must therefore be exercised in distinguishing between dead loads and live loads to prevent wasteful overdesign or dangerous underdesign.

Dead loads are defined here as *those permanent loads that are so rigidly fixed to the structural frame that in an earthquake, they will be accelerated at the same rate as the structural frame*. It is not important whether or not the structure is actually subject to earthquakes. It is important only that a consistent and accurate means of distinguishing which loads on a structure are to be classified as dead loads and which are not. The foregoing definition satisfies that requirement.

Mechanical equipment, ductwork, cable trays, carpets and other such loads are not classified as dead loads. Although they are indeed permanently present and immobile, they are subject to renewal, replacement, or alteration during the life of the building. In addition, they are not usually attached directly to the building frame; one cannot be sure that they will be accelerated at the same rate as the building frame.

Dead loads are those due to the weight of the building components. They may be a part of the structure (beams, frames, columns) or of appurtenances (firewalls, parapets, equipment housings). Dead loads consist of fixed definable objects having known computable weights. Dead loads cannot be changed during the life of the building except by additional construction or by remodeling.

A brief list of weights for some of the more common building materials has been compiled from various sources and is presented in Table 2-1. These weights may be used to compute the dead load of various objects. The list is by no means complete; more comprehensive lists of dead load weights are given in standard references[4,41].

In contrast to a dead load, *a live load is defined as any gravity load that is not accelerated at the same rate as the structural frame*. Live loads are, by default, the loose gravity loads left over after all dead loads have been identified. Only a few of the live loads on a structure can be accurately determined, however. Most live loads are the vaguely defined transient loads that move into, onto, or within the building during its service life.

Alternatively, live loads may be regarded as those that may be expected to occur due to the usage or occupancy of the structure. Over the service life of the structure, however, the usage of the structure can change sharply as tenants change or as new owners subject the structure to new functions. Any new loads must, of course, be kept within the original design limits, regardless what the new function is.

Table 2 - 1 WEIGHTS OF COMMON BUILDING MATERIALS

Material	psf	Material	psf
Ceilings		Partitions	
Furred channel system	1	Clay tile	
Acoustic fiber tile	1	3 in.	17
Floors		4 in.	18
Concrete, per inch		6 in.	28
Stone	12.5	8 in.	34
Slag	11.5	10 in.	40
Lightweight	6 to 10	Gypsum Block	
Fills, per inch		2 in.	9.5
Gypsum	6	3 in.	10.5
Sand	8	4 in.	12.5
Cinders	4	5 in.	14.0
Finishes, per inch		6 in.	18.5
Terrazzo	13.0	Plaster, per inch	
Quarry Tile	12.5	Cement	10
Mastic	11.5	Gypsum	5
Hardwood	5.0	Lathing	
Softwood	4.0	Expanded metal	0.5
Roofs		Gypsum board, 0.5 in,	2.0
Copper	1.0	Walls	
3-ply felt and gravel	5.5	Brick	
5-ply felt and gravel	6.0	4 in.	40
Shingles		8 in.	80
Wood	2	12 in.	120
Asphalt	3	Hollow concrete block	
Clay tile	9 to 15	Heavy aggregate	
Slate, 0.25 in.	10	4 in.	30
Sheathing, per inch		6 in.	44
Wood	4	8 in.	56
Gypsum	4	12 in.	80
Insulation, per inch		Light aggregate	
Loose	0.5	4 in.	21
Poured-in-place	2.0	6 in.	30
Rigid	1.5	8 in.	38
Corrugated asbestos		12 in.	56
0.25 in. thickness	3	Clay tile	
		4 in.	25
		6 in.	30
		8 in.	33
		12 in.	45
		Structural Glass, 1 in.	15

TABLE 2 - 2 MINIMUM DESIGN LIVE LOADS

a. Uniformly distributed live loads based on occupancy

Occupancy	psf	Occupancy	psf
Armories and drill rooms	150	Residential	
Assembly halls, theatres		Multifamily units	
Fixed seats	60	Private apartments	40
Movable seats	100	Public rooms	100
Balconies, exterior	100	Corridors	60
Bowling alleys	75	Single-family units	
Dance halls	100	First floor	40
Dining areas	100	Habitable attics	30
Gymnasium floors	100	Storage areas	30
Hospitals		Hotels	
Operating rooms	60	Guest rooms	40
Private rooms	40	Public rooms	100
Wards	40	Primary corridors	100
Libraries		Public corridors	60
Reading rooms	60	Private corridors	40
Stack rooms	150	Schools	
Manufacturing	125	Classrooms	40
Marquees	75	Corridors	100
Office Buildings		Stairs and fire escapes	100
Offices	80	Stands and Bleachers	100
Lobbies	100	Stores, retail	
Penal institutions		First-floor rooms	100
Cell blocks	40	Upper floors	75
Corridors	100	Stores, wholesale	125

b. Uniformly distributed live loads based on use

Use	psf	Use	psf
Air-conditioning equipment	200	Garages, ramps, drives	
Amusement-park structures	100	Trucks, 3 to 10 tons	150
Bakeries	150	Trucks, above 10 tons	200
Boiler rooms, framed	300	Hangars	150
Broadcasting studios	100	Incinerator floors	100
Catwalks	25	Kitchens, commercial	150
Dormitories		Laboratories, science	100
Partioned	40	Laundries	150
Nonpartitioned	80	Libraries, corridors	100
Fan rooms	150	Public rooms	100
File rooms		Rest rooms	60
Letter	80	Rinks, ice skating	250
Card	125	Storage, hay or grain	300
Addressograph	150	Telephone exchanges	150
Foundries	600	Toilet rooms	60
Fuel rooms, framed	400	Transformer rooms	200

In general, the live load used in structural design is taken as the load per square foot of floor area that may be expected during the service life of the structure. There is no attempt to identify specific items of load when one is establishing the live load. Rather, a blanket load is adopted which has been found in the past to provide satisfactory results without requiring wasteful overdesign.

For a starting point, ranges of live load which include a reasonable latitude for future changes are prescribed by the various building codes[21,35]. A brief list of some of the more commonly used live loads has been compiled from various sources and is presented in Table 2-2. These ranges of live load are based on rather broad categories for the intended usage. Over the years, they have been found by the industry to be generally satisfactory.

Distribution of Gravity Loads to Foundations

The magnitude of the vertical gravity load to be supported by each footing in a group of footings can be established readily, based on known and accepted methods of structural analysis. In the design methods presented in later chapters, it will be found that there will be no need to find the moment on an individual footing, which simplifies the procedure considerably. It should be apparent that the overall accuracy of any design method is limited by the accuracy of the loads. For live loads, such accuracy has been shown to be quite arbitrary.

For routine regular framed structures or buildings, the distribution of loads to individual footings may be determined using the following assumptions:

- The vertical gravity loads tributary to a particular footing may be computed with reasonable accuracy as the sum of all gravity loads halfway to the next adjacent vertical support in all directions.
- The vertical gravity loads tributary to a particular footing includes the loads at all floor levels and includes the entire load from any cantilevers that are tributary to the support.
- Any redistribution of gravity loads within a structure due to flexural moments can be neglected.

Verification of the foregoing assumptions may be found in the ACI approximate method for final design of frames braced against sidesway[2]. The ACI approximate method provides an acceptably accurate method of analysis for regular structures of any material. The ACI method does not require any redistribution of vertical load due to flexure. The ACI method is developed analytically and is accurate within its prescribed limitations.

The following limitations apply to the ACI approximate method and, consequently, to the simplified analysis presented here:

- The structure or building must be reasonably regular, that is, adjacent span lengths may not differ by more than 20%.
- Loads must be relatively uniformly distributed.
- Live load may not exceed dead load by more than three times the dead load.
- The structure or building must be framed from prismatic members, although it is expected that shear panels, braced panels, or bearing walls will be used in the lateral bracing system.

In the simplified approach, intuitive methods are usually reliable. A concentrated load, for example, can be distributed to its supporting members in proportion to its proximity to each support. Similarly, it may be intuitively assumed that a slab on grade makes no significant contribution to footing loads.

Municipal building codes sometimes require that floor beams be able to sustain a randomly placed concentrated load of 2000 pounds at any point in the span. The end result of this provision is to provide a minimum capacity in shear for all floor beams. The load itself is a part of the floor live load and is not intended to be added to each beam; it should not be included when computing column loads.

Also, municipal building codes sometimes permit a reduction in live loads when tributary areas are large. Too, some additional reductions are sometimes permitted in multistory buildings due to the improbability that all stories will be loaded to maximum load at the same time. Such refinements are indeed worthwhile in final design, but for the sake of simplicity they are not included here.

Example calculations follow, showing the computation of vertical design loads on footings both for dead load and for live load. A simple four story building is used for this example; the layout is that shown in Fig. 2-1.

Example Calculations of Gravity Loads on Footings

The following loads will be used in subsequent calculations.

Commercial office building, seismic zone 2A, wind velocity 100 mph
CH soil, shear strength 1250 psf
Roof live load 30 psf, floor live load 60 psf, stairwell live load 100 psf
Roofing: built-up roofing, weight 15 psf
Wall around mechanical equipment housing 600 plf
Rooftop equipment room: allow 150 psf for equipment
Floor surface: terrazzo concrete, weight 20 psf
Perimeter walls: masonry and glass, 650 plf per story
Shearwalls: concrete, 1400 plf per story
Interior partitions: movable partitions, allow 12 psf

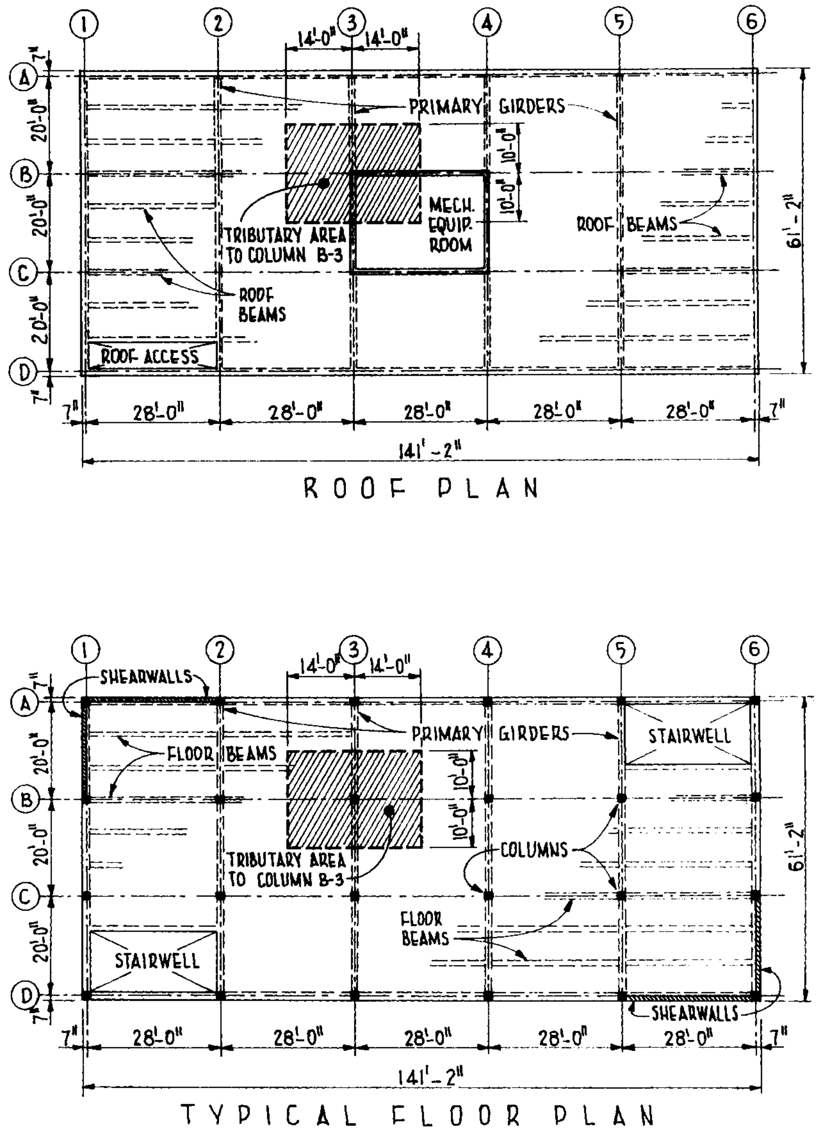


Figure 2-1 Typical Building Configuration

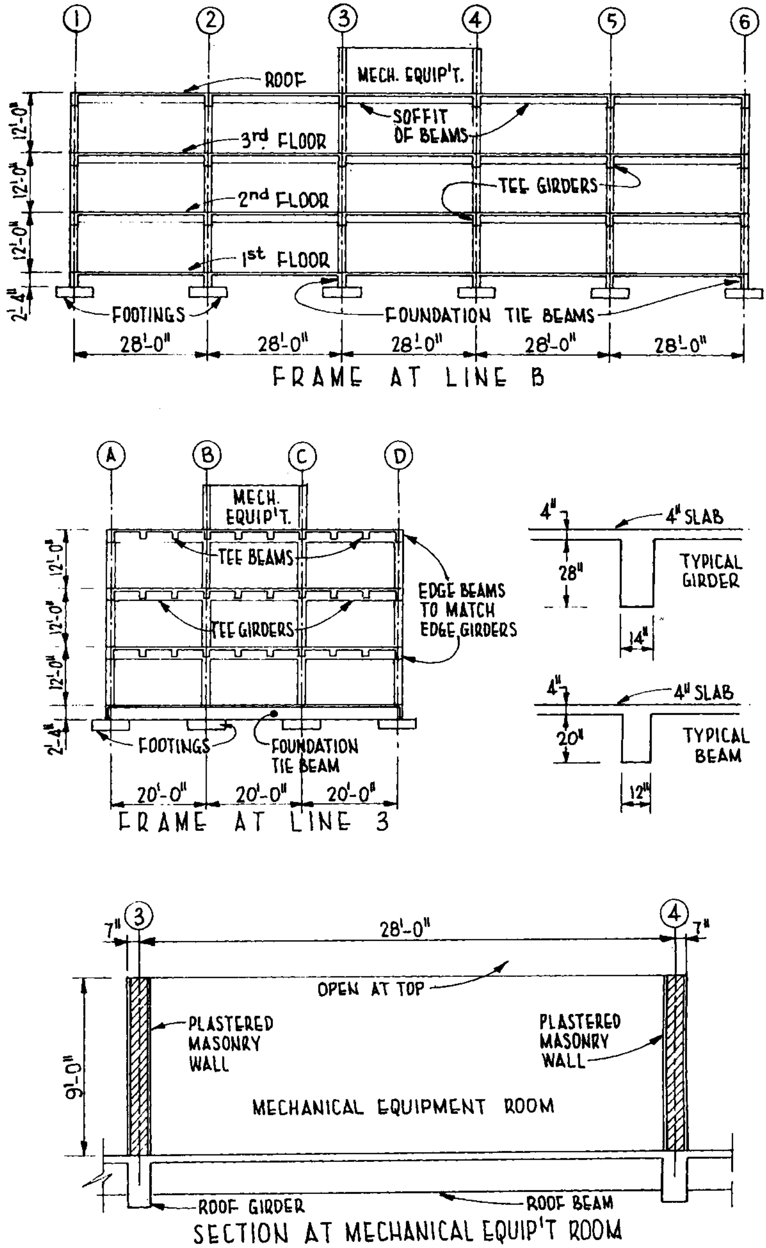


Figure 2-1 Typical Building Configuration (continued)

Example 2-1 Computation of load on a foundation

Given : Building and conditions shown in Fig. 2-1

To find: Dead loads on a typical interior column and footing

Solution:

The dead load tributary to the footing at grid point B3 is determined in the following calculations. It is noted that lateral loads are taken by shearwalls at corners A1 and D6. Consequently, footing B3 does not carry any lateral load.

For the sake of the computations, the weight of the floor and roof beams will be distributed uniformly over the floor. Within one 20 ft x 28 ft bay, there are three such beams, each 26 ft 10 in. long, with a stem 20 in.x 12 in.

$$Wt_{stem} = \frac{12 \times 20}{144} \times 26.83 \times 145 \times 3 = 19.5 \text{ kips}$$

Distributed uniformly, the stem weight becomes

$$w_{stem} = \frac{19{,}500}{20 \times 28} = 35 \text{ psf}$$

For the main girders, the load per foot for the stem, 28 in.x 14 in., is given by

$$\text{wt./ ft.} = \frac{28 \times 14}{144} \times 145 = 395 \text{ plf}$$

The tributary areas to column B3 are shown as the hatched areas in Fig. 2-1. The following calculations include all dead loads in those areas (in kips). A general value for the weight of concrete is taken as 145 pounds per cubic foot.

At roof level (including mechanical equipment room):

Wt. of wall = 600plf x 28/2 + 600plf x 20/2	= 14.4 kips
Wt. of roofing = 15psf x 20 x 28	= 8.4 k
Wt. of slab = (4/12)(145pcf)(20 x 28)	= 27.1
Wt. of beams = (35psf)(20 x 28)	= 19.6 k
Wt. of girder = 395plf x 18.83ft	= 7.4 k
Wt. of column = (14 x 14/144)(9.33ft)(145)	= 1.8 k

At third floor level:

Wt. of terrazzo = (20psf)(20 x 28)	= 11.2 k
Wt. of slab = (4/12)(145pcf)(20 x 28)	= 27.1 k
Wt. of beams = (35psf)(20 x 28)	= 19.6 k
Wt. of girder = 395plf x 18.83ft	= 7.4 k
Wt. of column = (14 x 14/144)(9.33ft)(145)	= 1.8 k

At second floor level:

Wt. of terrazzo = (20psf)(20 x 28)	= 11.2 k
Wt. of slab = (4/12)(145pcf)(20 x 28)	= 27.1 k
Wt. of beams = (35psf)(20 x 28)	= 19.6 k
Wt. of girder = 395plf x 18.83ft	= 7.4 k
Wt. of column = (14 x 14/144)(11.7ft)(145)	= 2.3 k
Total dead load to footing	= 214 k

It should be noted in Example 2-1 that door openings, floor recesses and minor deviations in dead load are ignored. Since the method itself is approximate, any undue refinement of loads is unwarranted. In fact, many designers would add an arbitrary 5% or 10% to the final sum to account for any items that may have been overlooked or to account for items that may be added in the future.

The next example shows the calculation of live loads tributary to the same footing.

Example 2-2 Computation of load on a foundation

Given : Building and conditions given in Fig. 2-1

To find: Live loads on a typical interior column and footing

Solution

Live load to the footing at grid point B3 is determined in the following calculations. The specified live loads to be used for design are stated in the problem data at the beginning of this section but are repeated here for immediate reference.

At the roof :	equipment load	= 150 psf
	live load	= 30 psf
At the floors:	live load	= 60 psf
	partition load	= 12 psf

The tributrary areas to column B3 for live load are identical to those for dead load, as indicated by the dashed lines in Fig. 2-1. The following calculations include all live loads within the boundaries (in kips)

At roof level:

Mechanical equip't load = (150)(20 x 28)/4	= 21.0 k
Roof live load = (30)(20 x 28)(3/4)	= 12.6 k

At third floor level:

Partition load = (12)(20 x 28)	= 6.7 k
Floor live load = (60)(20 x 28)	= 33.6 k

At second floor level:

Partition load = (12)(20 x 28)	= 6.7 k
Floor live load = (60)(20 x 28)	= 33.6 k

At ground level: all live loads carried on grade

Total live load to footing	= 114.2 k

The tributary dead loads and live loads calculated in Examples 2-1 and 2-2 are the gravity loads to be sustained at grid point B3. It should be noted that such calculations are direct and simple, with little refinement. Their accuracy, however, is within the same range of accuracy as the loads themselves.

Combinations of Gravity Loads

When no lateral loads occur, the maximum gravity load that can occur on a footing is simply the sum of the maximum dead load and maximum live load, denoted DL + LL in all subsequent discussions. The soil must have enough strength to support this maximum load, regardless how infrequently it may occur.

Another consideration in foundations, however, is the settlement of a foundation with time. It will be shown in later chapters that settlement in a soil is relatively unaffected by short-term loads; it is only the long-term sustained load on a footing that produces settlements. It thus becomes necessary to estimate how much of the transient live load will be in place long enough to affect footing settlements.

It should be apparent that not all live loads are in place long enough to affect settlements. Certainly, some portion of the live load such as furnishings, bookcases, carpets, cabinets, files, and so on, are in place long enough to produce settlements. Transient loads, including wind and earthquake, come and go with no effect on settlements: only the elastic deformations will occur under these short-term transient loads.

It is the usual practice to assume that the dead load plus some portion of the live load are in place long enough to produce settlements. For buildings serving routine architectural functions, the load causing settlements is commonly assumed to be the dead load plus about 50% of the live load. For buildings such as libraries, the long-term live load could be as much as 75% and for auditoriums as little as 25%. The actual percentage of live load to be used is thus a matter of judgment. The usual limits are somewhere between 20% and 80%. For the sake of simplicity, all subsequent calculations will use dead load plus 50% live load

The term "sustained load" is used herein to designate this long-term load. Other references sometimes call it the "reduced" load[31]. An example will illustrate the use of the sustained load in selecting the required size of the contact area.

Example 2-3 Design loads for footings

Given : Results of Examples 2-1 and 2-2

To find: Vertical Gravity loads to be used for footing design

Solution:

For peak load, DL + LL = 214 + 114 = 328 kips.

Under peak load, the maximum allowable bearing pressure for strength, p_a, may not be exceeded.

For sustained load, DL + 50%LL = 214 + 57 = 271 kips.

Under this load, the maximum allowable bearing pressure to limit settlements, p_a'', may not be exceeded.

Summary of Gravity Loads on Footings

In summary, footings are designed to support the vertical gravity loads under the following conditions:

- Maximum (or peak) vertical gravity loads DL + LL must be supported without exceeding the allowable bearing pressure for strength, p_a.
- Sustained (or long-term) vertical gravity loads DL + 50%LL must be supported without exceeding the allowable bearing pressure for settlements, p_a''.

Review Questions

2.1 What are the two broad categories of loads on structures?

2.2 In practice, how are dynamic loads usually handled in structural analysis and design?

2.3 What are the two design conditions that must be met when designing for gravity loadings?

2.4 Why is a reduced live load used when a foundation is being designed to meet settlement limitations?

2.5 Name three gravity loads in a typical building (other than components of the building) that would be classed as dead load.

2.6 Name three transient loads in a typical building that will be in place long enough to affect settlements.

2.7 Define a dead load.

2.8 Define a live load.

2.9 Under what circumstances could a live load be transformed into a dead load?

2.10 Why aren't live loads specifically listed and their weights determined in the same way dead loads are?

2.11 How is the gravity load on a specific footing determined?

2.12 What is the "ACI approximate method"?

2.13 What limitations in a building configuration must be met if the ACI method is to be applicable?

2.14 Define the "tributary area" for a footing load.

2.15 Would an air conditioner mounted on vibration isolators on the roof of a building be classified as a dead load or a live load?

2.16 Would a heavy fireproof file cabinet be classified as a dead load or a live load?

OUTSIDE PROBLEMS

2.1 Determine the dead load on footing A-3 of Fig. 2-1 using the design criteria preceding Example 2-1.

2.2 Determine the dead load on footing A-5 of Fig. 2-1 using the design criteria preceding Example 2-1. Use a dead load of 175 psf over the entire stairwell to account for stairs and landings.

2.3 Determine the dead load on footing A-6 of Fig. 2-1 using the design criteria preceding Example 2-1. Use a dead load of 175 psf over the entire stairwell to account for stairs and landings.

2.4 Determine the dead load on footing B-1 of Fig. 2-1 using the design criteria preceding Example 2-1.

2.5 Determine the dead load on footing B-5 of Fig. 2-1 using the design criteria preceding Example 2-1. Use a dead load of 175 psf over the entire stairwell to account for stairs and landings.

2.6 Determine the dead load on footing B-6 of Fig. 2-1 using the design criteria preceding Example 2-1. Use a dead load of 175 psf over the entire stairwell to account for stairs and landings.

2.7 Determine the live load on footing A-3 of Fig. 2-1 using the design criteria preceding Example 2-1.

2.8 Determine the live load on footing A-5 of Fig. 2-1 using the design criteria preceding Example 2-1. Use a live load of 100 psf over the entire stairwell to account for stairs and landings.

2.9 Determine the live load on footing A-6 of Fig. 2 -1 using the design criteria preceding Example 2-1. Use a live load of 100 psf over the entire stairwell to account for stairs and landings.

2.10 Determine the live load on footing B-1 of Fig. 2-1 using the design criteria preceding Example 2-1.

2.11 Determine the live load on footing B-5 of Fig. 2-1 using the design criteria preceding Example 2-1. Use a live load of 100 psf over the entire stairwell to account for stairs and landings.

2.12 Determine the live load on footing B-6 of Fig. 2-1 using the design criteria preceding Example 2-1. Use a live load of 100 psf over the entire stairwell to account for stairs and landings.

Chapter 3

LATERAL LOADS ON FOUNDATIONS*

Types of Lateral Loads

The design of structures for lateral loads is heavily dominated by building codes. The procedures and requirements prescribed by the codes are numerous and complex, encompassing many types of structures and their many variations. The procedures presented here are only a small "bite-size" chunk of this very large and complex subject.

The procedures given here are extracted from the *Uniform Building Code*, 1997 edition, with permission of the copyright holder, the International Congress of Building Officials. The focus of the code is the design of the entire structure, while the focus of this text is only on the design of the foundations. Consequently, code applications in this text are quite limited.

Soils are designed at working levels of stress, not at ultimate levels of load. However, under the procedures given by the *Uniform Building Code*, the earthquake load is determined for the superstructure at ultimate levels of load. To be applied to soils at working levels of stress, the load so obtained must be divided by its load factor of 1.4.

The type of structures that are likely to be designed with shallow foundations are low, relatively rigid structures. In general, these structures are less than 65 feet tall and have natural periods of oscillation less than 0.7 seconds. Fortunately, these same structures fall into a special category in the design codes in which simplified procedures are used. It is these simplified procedures that apply to foundation design and which are used here.

Lateral loads come from two sources: wind (denoted W) and earthquake (denoted E). Each of these loads has a distinctly different effect on the structure. At the foundation, however, the end result in both cases is a lateral force on the foundation as indicated in Fig. 3-1. A second consequence of lateral loading is an

* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

overturning moment on the structure which inherently produces an increase or decrease in vertical load on some of the footings.

Winds try to move a structure laterally across the earth's surface. Earthquakes try to move the earth laterally underneath the structure. In both cases, the result is a shear at the base of the structure as shown in Fig. 3-1. The acting shear and the resisting shear always form a couple, as indicated. The acting shear is called *base shear*. The couple moment is called the *overturning moment*. The *center of lateral forces* lies at h_{LAT} above the base.

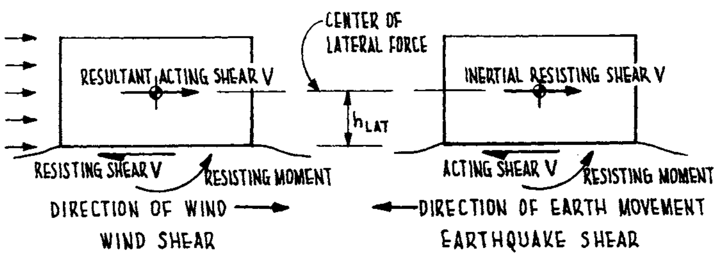


Figure 3 - 1 Base Shear under a Structure

In general, wind and earthquake loads are of such short duration that they do not produce settlements. They are therefore of primary concern where the allowable bearing pressure on the soil is governed by the strength of the soil rather than by its susceptibility to settlements.

With regard to settlements, only the differential settlements between footings are of interest here. The vibration of sandy soils produced by wind or earthquake might possibly result in settlements, but such settlements are more likely to be area-wide settlements than differential settlements between footings. Such area-wide settlements may have deleterious effects on utilities connections to the structure but they have little effect on the structure itself.

Stability under Combined Loading

For the gravity loads of Chapter 2, there are two limitations to be met: strength and settlement. For the lateral loads of this chapter, however, only the strength of the soil is of concern. Both wind and earthquake loads are so transient that no appreciable differential settlements occur.

Whenever lateral loads do occur, however, there will undoubtedly be gravity loads already in existence on the structure. The total load therefore will include both gravity loads and lateral loads. The maximum vertical load case is generally taken to be the sum of the maximum gravity loads plus any vertical loads that may have been induced by the maximum lateral loads. At working levels, the load cases are:

For wind: DL + LL + W

For earthquake: DL + LL + E/1.4

Besides creating additional vertical load, both wind and earthquake will cause lateral forces on some or all of the footings in a structure. The lateral force at each footing must be resisted by friction at the founding line, which creates horizontal shear stresses in the underlying soil. Such a load case is shown in Fig. 3-2; state of stress in the underlying soil is shown both with and without lateral loads.

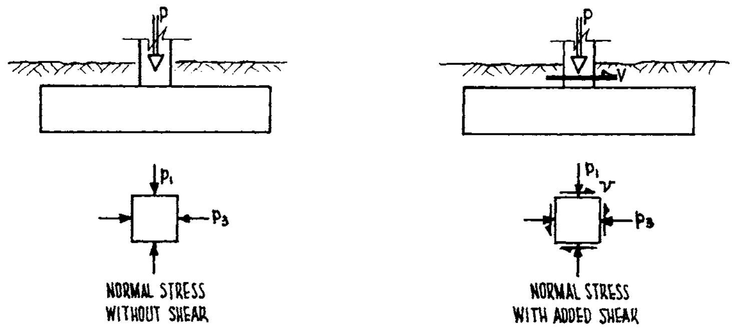


Figure 3-2 State of Stress under a Loaded Footing

Whenever a soil is subjected to combined vertical and lateral loads as shown in Fig 3-2, an allowable stress p_a' must be determined which reflects the combined state of stress. For this combined load case, a 33% increase in soil pressure p_a' is permitted when maximum lateral load occurs, but *the increase applies only to the transient components*. The combined load case then becomes either [DL + 0.75(LL + W)] or [DL + 0.75(LL + E/1.4)]. A moment's reflection will affirm that designing a footing for 75% of the maximum transient load will produce a 33% overstress attributable to the transient components when 100% of these loads occurs.

The lateral loads are resisted by friction at the base of the foundation: friction force $F = \mu N$, where N is the normal force and μ is the coefficient of friction. The worst-case load occurs when the gravity loads are lowest, which produces the lowest normal force. The worst-case loading for lateral frictional resistance becomes (DL + W) or (DL + E/1.4).

Lateral loads produce an external overturning moment. The worst-case resisting moment provided by the foundations also occurs when gravity loads are lowest. Consequently, the worst-case loading for resisting moment is also that when the gravity loads are least, (DL + W) or (DL + E/1.4).

The following points summarize the foregoing design features for shallow foundations under lateral load.

- Ultimate load design is not used in soils; shallow foundations are designed at working levels of stress.
- The allowable bearing pressure p_a' at working levels applies to combined stresses, normal plus shear.
- The foundation must sustain the maximum combined vertical loads without exceeding the allowable working bearing pressure p_a' for combined stresses:

  for wind: DL + 75%(LL + W)
  for earthquake: DL + 75%(LL + E/1.4),

- The structure must provide adequate resistance to sliding and adequate resistance to overturning at a time when it is under minimum load:

  for wind: DL + 75%W
  for earthquake: DL + 75%E/1.4

  For wind loads, Code[21] requires that the restoring moment exceed the overturning moment by at least 50%

Wind Velocities and Stagnation Pressures

The wind force on a structure is obviously proportional to the wind velocity. Wind velocity, in turn, is highly dependent on altitude, geography, terrain and local obstructions. The maximum wind velocity to be expected at a particular building is therefore quite difficult to determine; it is very sensitive to site conditions.

On a broad geographical basis, wind velocity charts have been created to show generally the wind velocities that may be expected in various regions[4,21,35,41] One such chart for the contiguous United States is shown in Fig. 3-3; there are others in use. The wind velocities shown in Fig. 3-3 are those to be expected at a height of about 33 feet (or 10 meters) above general ground surface.

Building codes generally classify structures by the amount of their exposure to unobstructed winds. The primary categories of exposure used in building design are described as:

- Exposure B has terrain with buildings, forest or surface irregularities, covering at least 20% of the ground level area extending one mile or more from the site.
- Exposure C has terrain which is flat and generally open, extending one-half mile or more from the site in any direction.
- Exposure D represents the most severe exposure in areas with basic wind speeds of 80 miles per hour or greater and has terrain which is flat and unobstructed facing large bodies of water over one mile or more in width relative to any quadrant of the building site.

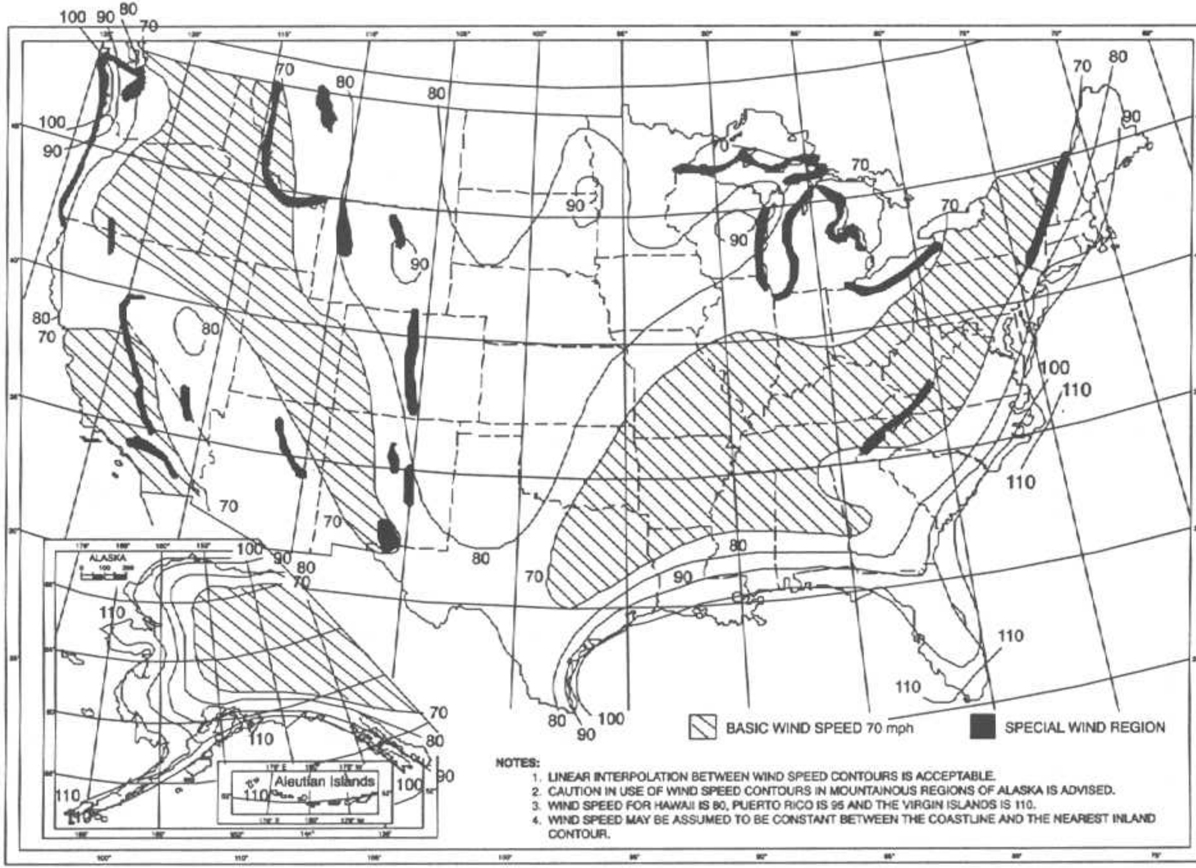


Figure 3-3 Basic Wind Speeds in Miles per Hour

[a]Reproduced from the 1997 edition of the *Uniform Building Code*, copyright 1997, with the permission of the publisher, the International Conference of Building Officials

A popular adaptation of these exposures is to take a rough average of Exposure B and Exposure C. Such a choice yields a wind load which can apply to most urban areas without excessive conservatism. For simplicity, this average is the exposure used throughout this text. It is defined mathematically in later discussions.

The effects of wind loads on low buildings or structures (less than about 65 feet high) are reasonably predictable. The wind pressure against the structure increases along the height of the building, being considerably less near the ground than at the top of the building. Typical pressure distribution against a building is shown in Fig. 3-4.

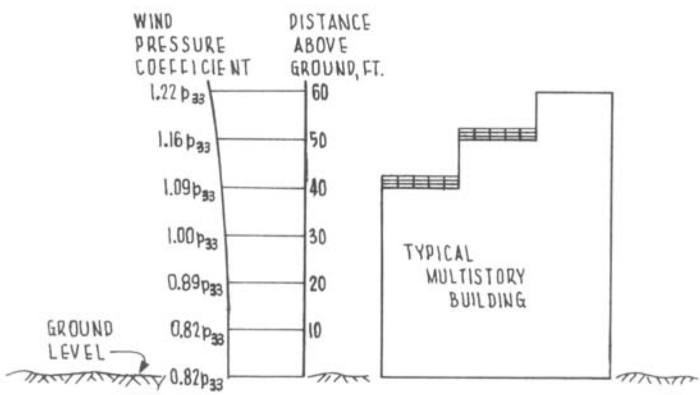


Figure 3-4 Typical Wind Pressure Distribution

Since the wind pressure varies along the height of the building, a reference *stagnation* wind pressure is chosen at the same standard height above ground level as the velocity, or 33 feet. The stagnation pressure is the pressure produced when all the kinetic energy in the wind ($\frac{1}{2}mv^2$) is dissipated into static conditions. At 33 feet above adjacent ground level, the stagnation pressure p_{33} is given by:

$$p_{33} = 0.00256V^2 \tag{3-1}$$

where V is the wind velocity in miles per hour, given in the wind velocity map of Fig. 3-3.

At heights other than 33 feet, the variation in wind stagnation pressure is computed by applying an empirical factor applied to the stagnation pressure p_s. For an approximate average of Exposure B and Exposure C, this empirical factor is given in the following equation for p_s:

$$p_s = 0.00256V^2\left[\frac{H}{33}\right]^{\frac{2}{7}} \tag{3-2}$$

where H is the height above ground level, with the minimum value of H being 15 feet. (Wind pressures below 15 feet are taken to be the same as the pressure at 15 feet.)

Shape Factors for Wind Loads

It is emphasized that the stagnation pressure given by Equation 3-2 is *not* the actual wind pressure against the building. To convert stagnation pressure to actual wind pressure, the stagnation pressure must be multiplied by a *shape factor*, which takes into account the shape of the structure and the effects of its shape on overall wind resistance. For the same stagnation pressures at 33 feet, for example, round cylinders offer much less resistance to wind loads than rectangular prisms.

Typical shape factors are given in Table 3-1 for various shapes of structures. These are the factors used to convert the stagnation pressures given by Equation 3-2 into actual wind pressures. It is well to note at this point that the shape factors of Table 3-1 take into account such things as cross-sectional shape, amount of openings and the inclination of the surface to the wind. They do not, however, include reveals, recesses or roughness of the exposed surface.

In a stability analysis, the wind "sees" the building as if it were a flat silhouette, projected horizontally onto a flat screen. Setbacks, reveals, recesses and other surface features of the building do not significantly affect the overall average wind pressure against the building. Surface roughness or irregularities have little effect on the average wind pressure.

In addition to the horizontal force, wind also produces a suction force on the roof, as reflected by the factors in Table 3-1. Its net effect on low buildings is to offset some or all of the roof live loads, if any in fact exist under high winds. It has been noted that the sustained load assumed to exist throughout the day-to-day service life of a structure is a very rough conservative guess of DL + 50%LL. Under such "iffy" approximations and assumptions, this further refinement of roof live load is generally ignored insofar as base shear and overturning moment are concerned.

TABLE 3-1 Shape Factors for Primary Frames and Systems

STRUCTURE OR PART THEREOF	SHAPE FACTOR
Primary frames and systems of rectangular prismatic structures	
On walls	
Windward wall	0.8 inward
Leeward wall	0.5 outward
On ridged roofs[a]	
Wind perpendicular to ridge	
Windward roof	
Slope < 1:6	0.7 outward
Slope ≥ 1:6 but < 3:4	0.9 outward, or 0.3 inward
Slope ≥ 3:4 but ≤ 1:1	0.4 inward
Slope > 1:1	0.7 inward
Leeward roof	0.7 outward
Wind parallel to ridge, entire roof	0.7 outward
On flat roofs	0.7 outward
Solid towers (chimneys, tanks, silos, etc)	
Square or rectangular	1.4 any direction
Hexagonal or octagonal	1.3 any direction
Round or elliptical	0.8 any direction
Open frame towers[b,c]	
Square and rectangular	
Diagonal elements	4.0
Normal elements	3.6
Triangular	3.2
Tower accessories (ladders, conduit, lights, etc)	
Cylindrical members	
2 inches or less in diameter	1.0
more than 2 inches in diameter	0.8
Flat or angular members	1.3
Minor[c] structures (signs, fences, billboards)	1.4 any direction
Poles[c] (flagpoles, lightpoles, etc.)	1.4 any direction

[a]All pressures are normal to the surface

[b]Wind pressures shall be applied to the total normal projected area of all elements on one face. The forces shall be assumed to act parallel to the wind direction.

[c]Factors for cylindrical elements are two thirds of those for flat or angular elements.

Calculation of Base Shear Due to Wind

The total horizontal force against the foundations, called the *base shear* and denoted V, is simply the accumulated sum of forces at the various levels. The wind force at each level i is computed as the average pressure p_i at that level times the area A_i subject to that pressure. Their sum is the base shear.

$$\text{Total applied force } V = \text{Base shear} = \sum p_i A_i \qquad (3\text{-}3)$$

As indicated in Fig. 3-5, the horizontal wind pressure is commonly taken in stepped increments rather than as a smooth curve. The pressure p_i at any level i can be computed by using Equation 3-2; the result is then multiplied by the appropriate shape factor to obtain the pressures p_i.

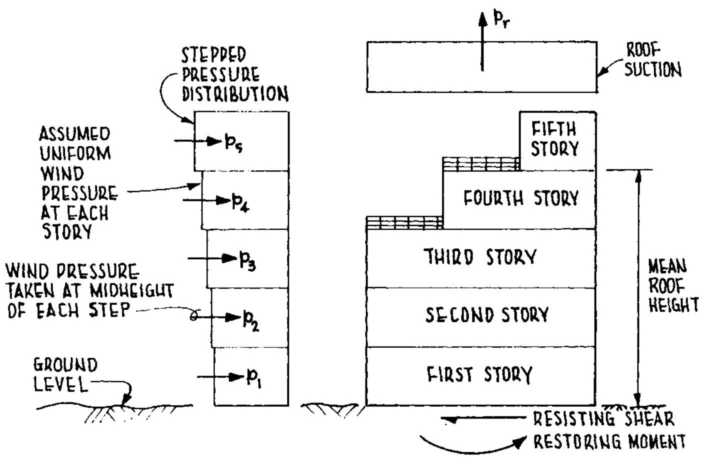


Figure 3-5 Typical application of wind pressure.

The roof suction force is computed as an uplift force over the entire roof area. As indicated in Fig. 3-5, the effective suction pressure is the stagnation pressure at mean roof height multiplied by the roof shape factor.

Overturning Moment Due to Wind

In addition to producing the base shear, the wind loads also produce an overturning moment on the entire building, thus creating additional vertical load on the foundations. The overturning moment is readily computed as the static moment of the wind forces acting on the building. The opposing moment, shown as the "restoring moment" in Fig. 3-5, is the resisting moment that must be developed by the foundations.

It is noted that the wind pressure against certain components in and around a building can be much higher than the average pressure, depending on localized wind turbulence and eddy currents. Such localized variations in the average pressure against the superstructure do not affect the overall base shear and overturning moment delivered to the foundations. Such refinements in the superstructure design are not pursued further here. Standard references present the effects of wind loads on the superstructure[4].

It is also noted that the building codes require that certain important buildings be designed for higher wind loads than other buildings in the area. Typical buildings that might fall in this category are civil emergency centers, hospitals, fire stations, police stations and communications centers. The increased wind load is determined by multiplying the wind base shear given by Equation 3-3 by an "importance factor" I. The importance factor can be as high as 1.15 for wind loads. For the sake of simplification, the importance factor is taken herein at its base value of 1.00.

The following example illustrates the calculation of the base shear and the overturning moment due to wind.

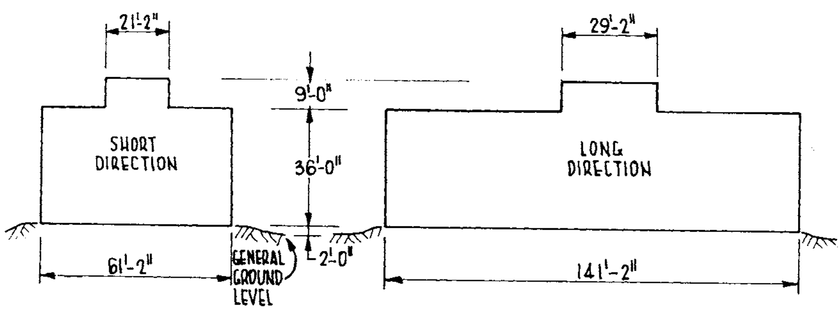
Example 3-1 Wind loads on a small building

Given : Building and conditions given in Chapter 2, Fig. 2-1 for a design wind velocity 100 mph

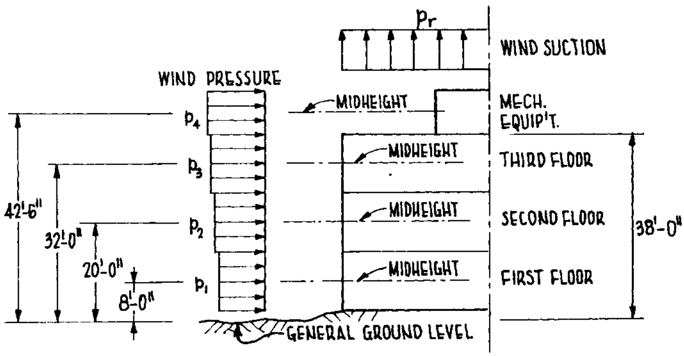
To find: Base shear V, uplift force F_u and overturning moment M_{ov}

Solution:

The base shear due to wind acting on the building of Fig. 2-1 will be computed for each of the two axes. The silhouettes of the two views of the building are shown in the following sketches. Pressures will be computed at midlevel of each floor and at mean height of the roof.



The variation of pressure along the height of the building is computed using Equation 3-2. This result is multiplied by the shape factor of 1.3 for prismatic shapes, as prescribed by Table 3-1. (For base shear and overturning, it does not matter that this pressure is actually distributed as 80% on the pressure side and 30% on the suction side; the total factor is 1.3.)



The pressure at any level in any direction is computed as the stagnation pressure times the shape factor. For a wind velocity of 100 mph, the pressures are:

At mean roof height:	$p_r = 26.6 \times 0.7$	$= 18.6$ psf
At mechanical room:	$p_4 = 27.5 \times 1.3$	$= 35.8$ psf
At third floor:	$p_3 = 25.4 \times 1.3$	$= 33.0$ psf
At second floor:	$p_2 = 22.2 \times 1.3$	$= 28.9$ psf
At first floor:	$p_1 = 20.4 \times 1.3$	$= 26.5$ psf

The total force at any level is computed simply as pressure times projected area. For wind pressures normal to the short side of the building, the forces at the various levels are (in kips):

At roof:	$F_r = 18.6 \times 61.2 \times 141.2$	= 161 k(up)
At mech. rm.:	$F_4 = 35.8 \times 21.2 \times 9$	= 6.8 k
At 3rd floor:	$F_3 = 33.0 \times 61.2 \times 12$	= 24.2 k
At 2nd floor:	$F_2 = 28.9 \times 61.2 \times 12$	= 21.2 k
At grnd floor:	$F_1 = 26.5 \times 61.2 \times 12$	= 19.5 k

Total base shear normal to the short side $V = 72$ k
Total uplift force at center of symmetry $F_u = 161$ k

The overturning moment M_{ov} about any transverse axis is the static moment of the wind forces just determined. (In this structure, the upward vertical force at the roof falls at the center of symmetry and makes no contribution to overturning moment.)

$$M_{ov} = 6.8 \times 42.5 + 24.2 \times 32.0 + 21.2 \times 20.0 + 19.5 \times 8 = 1640 \text{ kip}\cdot\text{ft}$$

The center of lateral force acts at a point h_{LAT} above ground level:

$$h_{LAT} = M_{ov}/V = 1640/72 = 22.8 \text{ ft above ground}$$

For wind pressures normal to the long side of the building, the forces at the various levels are:

At roof:	$F_r = 18.6 \times 61.2 \times 141.2$	$= 161$ k(up)
At mech. rm.:	$F_4 = 35.8 \times 29.2 \times 9$	$= 9.4$ k
At 3rd floor:	$F_3 = 33.0 \times 141.2 \times 12$	$= 55.9$ k
At 2nd floor:	$F_2 = 28.9 \times 141.2 \times 12$	$= 49.0$ k
At grnd floor:	$F_1 = 26.5 \times 141.2 \times 12$	$= 44.9$ k

Total base shear normal to the long side $V = 159$ k

Total uplift force at center of symmetry $F_u = 161$ k

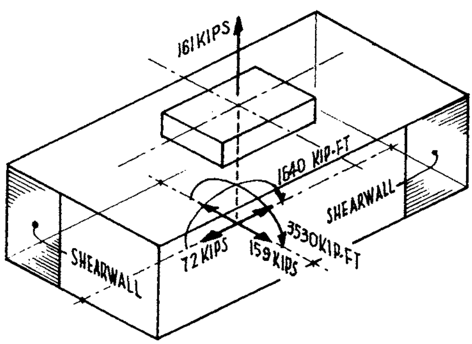
The overturning moment M_{ov} about any longitudinal axis is the static moment of these wind forces:

$$M_{ov} = 9.4 \times 42.5 + 55.9 \times 32.0 + 49.0 \times 20.0 + 44.9 \times 8 = 3530 \text{ kip•ft}$$

The center of lateral wind forces acts at a point h_{LAT} above ground level:

$$h_{LAT} = M_{ov}/V = 3530/159 = 22.2 \text{ ft above ground level}$$

The distribution of these base shears and overturning moments on the foundation is presented in later examples. These loads, which may come from either direction, are shown in the following sketch.



Earthquake Loads on Structures

Earthquake loads are distinctly different from wind loads. Wind forces are pressure forces, created at the exterior surfaces of a structure by a moving air mass. In contrast, earthquake forces are inertia forces, created at every molecule of mass in every member of the structure as the structure is being shaken by earthquake motions. The effects of the two types of loads within the superstructure are obviously quite different.

An earthquake creates both lateral motions and vertical motions in a structure. Under Newton's second law relating force F, mass m and acceleration a ($F = ma$), it is the rate of acceleration of these motions that governs the magnitude of the earthquake forces. In general, vertical earthquake motions can produce vertical inertia forces as high as 20% of the dead load, acting either upward or downward. Similarly, lateral earthquake motions can produce lateral inertia forces as high as 30% or even 40% of the dead weight of the building, acting laterally in any direction.

Structures are typically designed for vertical gravity loads of 100% dead load plus 100% live load, with a nominal margin of safety of roughly 70% to failure load. Consequently, the additional vertical load created by an earthquake (as much as 25% of dead load) is not regarded as a serious overload. In general, the vertical load produced by an earthquake is considered to be within acceptable limits for a one-time load and no special measures are taken to provide for it.

The base shear created by earthquake forces on a structure is an inertia force. In earthquake design, this inertia force is computed as a factor times the dead weight of the structure. Building codes[21,35] require up to 25% of live load be added to the dead load under some circumstances.

The rationale for using primarily dead load in the calculation of earthquake forces bears repeating. The only loads that can contribute to base shear are the loads that will be accelerated by the earthquake motions. In the classifications of Chapter 2, it was shown that live loads are loose, or at least so loosely fastened that they will not be accelerated at the same rate as the structural frame. Even a small amount of slippage of an object will reduce the inertia force so sharply that it will make very little contribution to base shear. Though only fixed loads produce inertia forces, codes do require a small percentage of live load be included with dead loads.

Seismic Risk Zones and Zone Factors

The location and intensity of major earthquake activity in the United States is now well charted, though an occasional surprise still occurs. An earthquake risk map is shown in Fig. 3-6, in which the regions of high, medium and low intensity of motion are shown as "seismic risk zones".

Risk zones are numbered from 0 to 4, with zone 4 being the zone of maximum earthquake risk and maximum earthquake intensity. Risk zones numbered 0 are essentially free of earthquake motions large enough to be a hazard to structures.

Code assigns numerical values called *zone factors* to the risk zones indicated in Fig. 3-6. Denoted Z, the values are:

$$\begin{array}{ll} \text{Risk Zone 1:} & Z = 0.075 \\ \text{Risk Zone 2A:} & Z = 0.150 \\ \text{2B:} & Z = 0.200 \\ \text{Risk Zone 3:} & Z = 0.300 \\ \text{Risk Zone 4:} & Z = 0.400 \end{array} \qquad (3\text{-}4a,b,c,d,e)$$

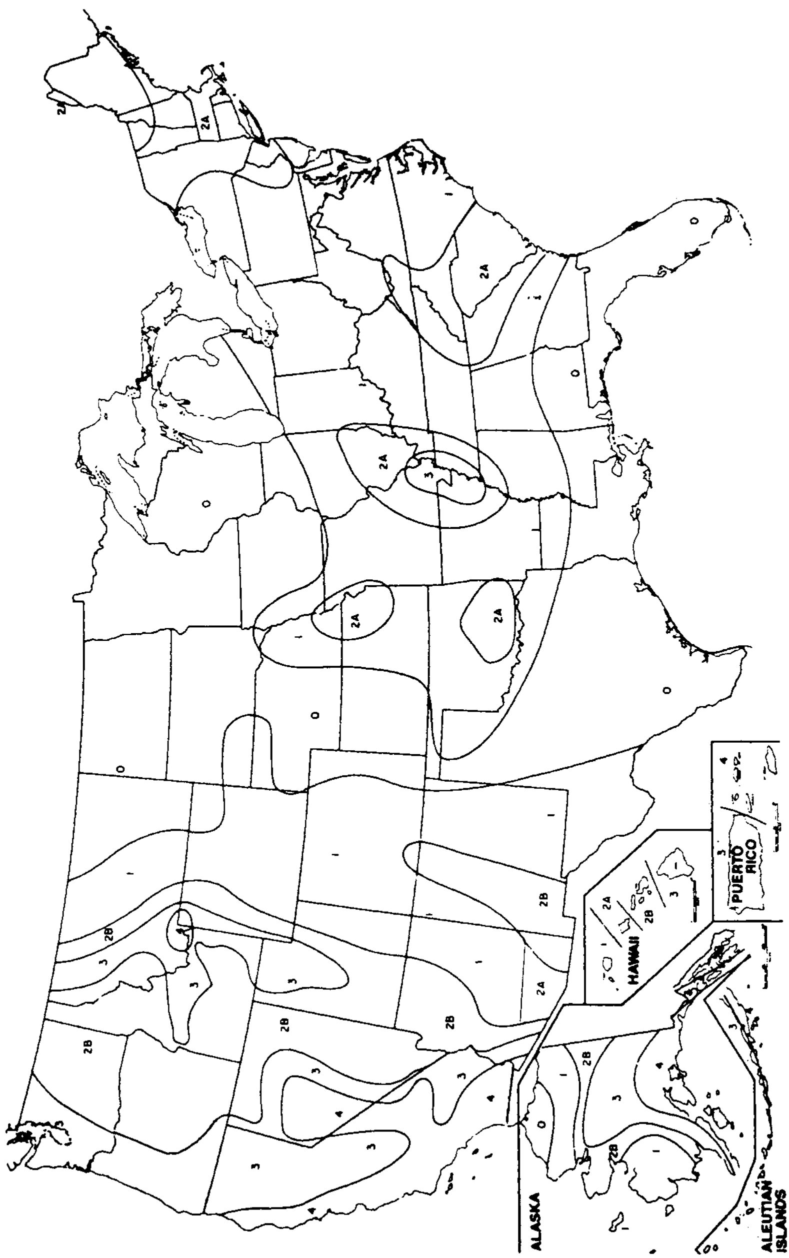


Figure 3-6 Seismic Zone Map of the United States
Reproduced from the 1997 edition of the *Uniform Building Code*, copyright 1997, with the permission of the publisher, the International Conference of Building Officials

Seismic Response of Building Systems

The magnitude of the earthquake inertia forces on a structure will vary with the natural period of oscillation of the structure. The type of structural system thus has a bearing on the magnitude of the inertia forces and the consequent base shear.

Codes separate the various structural systems in common use into groups that have similar responses. Each system is assigned a response factor R. A listing of some of the structural systems and their response factors R have been extracted from the Code and are presented in Table 3-2.

The response factors R listed in Table 3-2 take into account the relative rigidity of the structural system. A very flexible structure will sway when subjected to motions at its base, thereby reducing the base shear considerably. In contrast, a low rigid building having a stiff structural system can actually undergo accelerations as much as $2^1/_2$ times as large as the ground accelerations.

The structural systems that are most likely to utilize shallow spread footings are the low diaphragm-shearwall structures listed in the first two categories of Table 3-2; in these structures, walls carry all lateral loads. Typically, these structures are low and rigid, having heights less than 65 feet and periods less than 0.7 secs.

The moment-resistant frames listed in the third category of Table 3-2 are more likely to be used for taller structures; they are not often competitive in cost for lower buildings.

Soil Profile Type for a Building Site

The magnitude of the earthquake inertia forces is also dependent on the type of soil at the site as well as its strength and its depth. Codes group these properties into a *soil profile type* for the site. The 1997 *Uniform Building Code* lists six such soil profile types, given in Table 3-3.

UBC 1997 treats the determination of the Soil Profile Type in rigorous detail in its Section 1636. The determination of both the blow count for the Standard Penetration Test and the shear strength of the soil are specified in detail by UBC. The evaluation of the soil in these computations extends the full depth of the top 100 feet of the soil. The strength tests are discussed further in Chapter 5.

Seismic Coefficient for a Structure

Based on the risk zone Z of a building site, the soil profile type for the site and the type of structural system to be used, an average *seismic coefficient* for the structure is defined by Code. Tables of values for two such coefficients C_v and C_a are given in Table 3-4 and Table 3-5, respectively.

Table 3-2* STRUCTURAL SYSTEMS

BASIC STRUCTURAL SYSTEM[2]	LATERAL-FORCE-RESISTING SYSTEM DESCRIPTION	R	Ω_o	HEIGHT LIMIT FOR SEISMIC ZONES 3 AND 4 (feet) × 304.8 for mm
1. Bearing wall system	1. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	5.5	2.8	65
	b. All other light-framed walls	4.5	2.8	65
	2. Shear walls			
	a. Concrete	4.5	2.8	160
	b. Masonry	4.5	2.8	160
	3. Light steel-framed bearing walls with tension-only bracing	2.8	2.2	65
	4. Braced frames where bracing carries gravity load			
	a. Steel	4.4	2.2	160
	b. Concrete[3]	2.8	2.2	—
	c. Heavy timber	2.8	2.2	65
2. Building frame system	1. Steel eccentrically braced frame (EBF)	7.0	2.8	240
	2. Light-framed walls with shear panels			
	a. Wood structural panel walls for structures three stories or less	6.5	2.8	65
	b. All other light-framed walls	5.0	2.8	65
	3. Shear walls			
	a. Concrete	5.5	2.8	240
	b. Masonry	5.5	2.8	160
	4. Ordinary braced frames			
	a. Steel	5.6	2.2	160
	b. Concrete[3]	5.6	2.2	—
	c. Heavy timber	5.6	2.2	65
	5. Special concentrically braced frames			
	a. Steel	6.4	2.2	240
3. Moment-resisting frame system	1. Special moment-resisting frame (SMRF)			
	a. Steel	8.5	2.8	N.L.
	b. Concrete[4]	8.5	2.8	N.L.
	2. Masonry moment-resisting wall frame (MMRWF)	6.5	2.8	160
	3. Concrete intermediate moment-resisting frame (IMRF)[5]	5.5	2.8	—
	4. Ordinary moment-resisting frame (OMRF)			
	a. Steel[6]	4.5	2.8	160
	b. Concrete[7]	3.5	2.8	—
	5. Special truss moment frames of steel (STMF)	6.5	2.8	240

N.L.—no limit

[1]See Section 1630.4 for combination of structural systems.

[2]Basic structural systems are defined in Section 1629.6.

[3]Prohibited in Seismic Zones 3 and 4.

[4]Includes precast concrete conforming to Section 1921.2.7.

[5]Prohibited in Seismic Zones 3 and 4, except as permitted in Section 1634.2.

[6]Ordinary moment-resisting frames in Seismic Zone 1 meeting the requirements of Section 2211.6 may use a R value of 8.

[7] Total height of the building including cantilevered columns.

[8]Prohibited in Seismic Zones 2A, 2B, 3 and 4. See Section 1633.2.7.

Table 3-3* SOIL PROFILE TYPES

SOIL PROFILE TYPE	SOIL PROFILE NAME/GENERIC DESCRIPTION	AVERAGE SOIL PROPERTIES FOR TOP 100 FEET (30 480 mm) OF SOIL PROFILE		
		Shear Wave Velocity, $\bar{v}_s$ feet/second (m/s)	Standard Penetration Test, $\bar{N}$ [or $\bar{N}_{CH}$ for cohesionless soil layers] (blows/foot)	Undrained Shear Strength, $\bar{s}_u$ psf (kPa)
S_A	Hard Rock	> 5,000 (1,500)	—	—
S_B	Rock	2,500 to 5,000 (760 to 1,500)		
S_C	Very Dense Soil and Soft Rock	1,200 to 2,500 (360 to 760)	> 50	> 2,000 (100)
S_D	Stiff Soil Profile	600 to 1,200 (180 to 360)	15 to 50	1,000 to 2,000 (50 to 100)
S_E[1]	Soft Soil Profile	< 600 (180)	< 15	< 1,000 (50)
S_F	Soil Requiring Site-specific Evaluation. See Section 1629.3.1.			

[1]Soil Profile Type S_E also includes any soil profile with more than 10 feet (3048 mm) of soft clay defined as a soil with a plasticity index, $PI > 20$, $w_{mc} \geq 40$ percent and $s_u < 500$ psf (24 kPa). The Plasticity Index, PI, and the moisture content, w_{mc}, shall be determined in accordance with approved national standards.

Table 3-4* SEISMIC COEFFICIENT C_v

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_v$
S_B	0.08	0.15	0.20	0.30	$0.40N_v$
S_C	0.13	0.25	0.32	0.45	$0.56N_v$
S_D	0.18	0.32	0.40	0.54	$0.64N_v$
S_E	0.26	0.50	0.64	0.84	$0.96N_v$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

Table 3-5* SEISMIC COEFFICIENT C_a

SOIL PROFILE TYPE	SEISMIC ZONE FACTOR, Z				
	$Z = 0.075$	$Z = 0.15$	$Z = 0.2$	$Z = 0.3$	$Z = 0.4$
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

[1]Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

Calculation of Base Shear Due to Earthquake

(It is well to recall that the procedures given by UBC for finding earthquake forces will yield the ultimate loads to be used in strength design. For use at elastic levels, the forces so obtained must be divided by the load factor 1.4.)

In recognition of all the foregoing influences, Code specifies the value of the design base shear V based on the *average* acceleration of the superstructure:

$$V = \frac{C_v I}{R\,T} W \tag{3-5}$$

where: C_v is an average seismic coefficient specified by Code, given in Table 3-4
I is the importance factor
R is the interactive response factor specified by Code, given in Table 3-2
T is the natural period of the structure
W is the dead weight of the structure

For simplicity in all following discussions, the importance factor I is again taken at its base value of 1.0, as it was in the discussions of wind loading.

For the low structures (about 65 feet or less) that are likely to be founded on shallow foundations, the calculation of the base shear can be simplified considerably over the calculations required for higher structures. In the low structures that are of primary interest here, the upper bound for the base shear is also specified by Code[21]:

$$V = 2.5\frac{C_a I}{R} W \tag{3-6}$$

where C_a is an average seismic coefficient given in Table 3-5

Code gives equation (3-6) as an absolute upper bound on all values computed from equation (3-5). For the low structures of interest here, the upper bound is found to be the applicable equation for most structures up to about 50 feet high, and is only slightly in error up to about 70 feet. In all cases, the error is on the "safe" side.

Adopting the upper bound for the design of routine shallow foundations provides a worthwhile simplification of the design procedure. It is noted that the natural period of the structure drops out of consideration. The omission of the natural period is not considered serious; its computation is only a broad approximation prescribed by Code. The use of the upper bound given by equation (3-6) is adopted here for all further calculations.

Experimentation with equation (3-6) reveals that for a low structure founded on shallow spread footings, the maximum earthquake force will be about 10% to 13% of the vertical dead load. In areas of light earthquake intensity, the lateral load may drop to as little as 2.5% of the dead load. These percentages of the vertical dead weight are known as the "lateral g-load" or "lateral g-force" on the structure.

Overturning Moment Due to Earthquake

Since the base of a structure is securely anchored to the ground, the base will undergo accelerations identical to the accelerations of the ground. The top of a structure, however, can undergo accelerations as much as $2^1/_2$ times that of its base. Such an amplification may be attributed either to the effects of partial resonance or the effects of "whip" or a combination of both.

Current building codes require that the accelerations (and thus the inertia forces) be assumed to increase linearly with height above the base. Codes also requires that the *average* acceleration which is used to compute the base shear must remain constant. The base shear is thus presumed to be a constant resultant force whose component inertia forces along the height of the structure are in continuous fluctuation.

The overall effect of inverting the acceleration rates (zero at the base, maximum at the top) is to increase markedly the inertia forces toward the top of the structure. This "whip" effect also increases significantly the overturning moment produced by these inertia forces. (For some two or three story structures, Code[21] does not require this inversion)

The component of the base shear to be assigned to any level x between levels from 1 to n is computed by multiplying the base shear V by the inertia factor C_x for that level, or,

$$F_x = C_x V \tag{3-7a}$$

$$\text{where} \quad C_x = \frac{w_x a_{avg} \dfrac{h_x}{h_n}}{\sum_{i=1}^{n} w_i a_{avg} \dfrac{h_i}{h_n}} = \frac{w_x h_x}{\sum_{i=1}^{n} w_i h_i} \tag{3-7b}$$

and where w_i = dead load weight at level i
h_i = height of level i above the base
h_n = height of the highest level
a_{avg} = average acceleration (at h_{LAT})

The overturning moment M_{ov} produced by these forces F_x is calculated as usual:

$$M_{ov} = \sum_{i=1}^{n} F_i h_i \tag{3-8}$$

The location of the *center of lateral inertia forces* above the base, h_{LAT}, is also calculated as usual,

$$h_{LAT} = \frac{M_{ov}}{V} \tag{3-9}$$

The following example demonstrates the calculation of the base shear and overturning moment on a typical small building.

Example 3-2 Earthquake loads on a small building

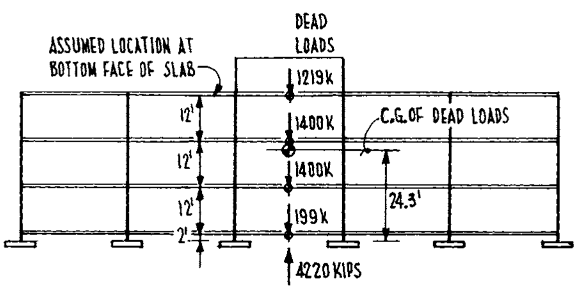
Given : Building and conditions given in Chapter 2, Fig. 2-1. Soil at the site is a deep alluvial clay having a shear strength of 1250 psf.

To find: Base shear and overturning moment due to earthquake motions

Solution:

The total dead weight of the building has been calculated aside using the methods of Chapter 2 and has been found to be 4220 kips. Center of gravity of dead load, h_{cg}, has also been calculated aside and found to be 24.3 ft. above top of footings.

The values of the dead loads at each floor level and the location of the center of gravity of all dead loads are shown in the following sketch. It is assumed that the resultant dead load of the individual floor loads are located approximately at the bottom surface of the slabs. Dimensions are shown from top of footing.



In succeeding computations, the base shear due to earthquake acting on the building of Fig. 2-1 is determined. It should be noted that the base shear is the same in either the long or short direction.

The base shear V is calculated using Equation 3-5, with the importance factor I taken as 1.00:

$$V = 2.5(C_a/R)W$$

The seismic risk zone for the building is given as risk zone 2A. The zone factor corresponding to risk zone 2A is found from equation 3-4b to be 0.15.

The soil profile is found from Table 3-3 for a seismic zone factor of 0.15 and a shear strength of 1250 psf. The profile is found to be S_D, indicating a stiff soil.

For S_D soil profile and a zone factor of 0.15, seismic coefficient C_a is found from Table 3-5 to be 0.22.

The building is identified as a building frame system with the concrete shearwalls taking only lateral load; columns take all the vertical load. From Table 3-2, the response factor R is found to be 5.5.

With a total dead load of 4220 kips, the base shear is:

$$V = 2.5 \times (0.22/5.5) \times 4220 = 422 \text{ kips}$$

The lateral inertia forces corresponding to the dead weights at each floor level are found at each floor diaphragm. Their sum is, of course, equal to the base shear and their resultant is located at the center of lateral inertia forces, at height h_{LAT}.

The inertia forces are calculated from equation (3-7a) and (3-7b). The denominator of equation (3-7b) is calculated as:

$$\sum_{1}^{n} w_i h_i = 1219 \times 38 + 1400 \times 26 + 1400 \times 14 + 199 \times 2$$

$$= 102720$$

At the roof diaphragm

$$C_x = \frac{1219 \times 38}{102720} = 0.451; \qquad \mathrm{F_R} = C_x V = 190 \text{ kips}$$

At the third floor diaphragm,

$$C_x = \frac{1400 \times 26}{102720} = 0.354; \qquad \mathrm{F_3} = C_x V = 149 \text{ kips}$$

At the second floor diaphragm,

$$C_x = \frac{1400 \times 14}{102720} = 0.191; \qquad \mathrm{F_2} = C_x V = 81 \text{ kips}$$

At the ground floor diaphragm,

$$C_x = \frac{199 \times 2}{102720} = 0.004; \qquad \mathrm{F_G} = C_x V = 2 \text{ kips}$$

The overturning moment is calculated as the static moment of these inertia forces about the top of the foundations:

$$M_{ov} = 190 \times 38 + 149 \times 26 + 81 \times 14 + 2 \times 2$$

$$= 12{,}200 \text{ kip} \bullet \text{ft}$$

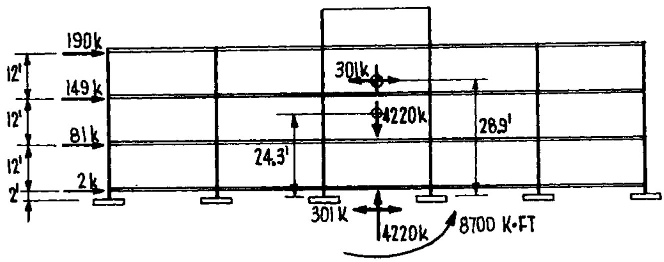
The base shear and overturning moment are now divided by the load factor 1.4 to obtain values at working levels,

$$M_{ov} = 12200 / 1.4 = 8700 \text{ kip} \bullet \text{ft}$$
$$V = 422 / 1.4 = 301 \text{ kips} \qquad \text{(two panels)}$$

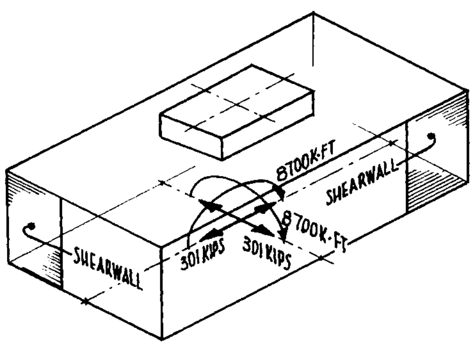
The center of lateral inertia forces above the base, h_{LAT}, is found as usual,

$$h_{LAT} = M_{ov} / V = 8700/301 = 28.9 \text{ ft.}$$

The final values of the base shear, the resisting shear and the resisting moment are shown in their correct positions in the following sketch.



The two sets of base shears and overturning moments at working levels of stress are shown in the following sketch. The inertia forces could come from either direction on either axis and are equal on both axes.



The results of the foregoing earthquake analysis can now be compared to the results of the wind analysis given in Example 3-1. It is observed that the base shears and overturning moments due to earthquake are considerably larger in both directions than the base shears and overturning moments due to wind. In the building of this example, the foundations would therefore be designed for lateral earthquake loads on both axes of the building.

Effect of Lateral Load on Footings of Rigid Frames

Regardless whether the highest lateral load is generated by wind or earthquake, the foundation system must sustain this lateral load. The applied lateral load acting on a structure is of course independent of the structural system. The external load on the structure is the same whether it is resisted internally by a braced frame or by a rigid frame.

For rigid frames, the distribution of lateral loads to individual columns and footings can be made using the *portal frame* analysis for rigid frames. In this method, the size and rigidity of the interior columns is assumed to be roughly twice that of the perimeter columns. Such an assumption is reasonably true in both directions for rigid frames having a regular column module.

Since the interior columns are roughly twice as rigid (their moments of inertia are twice as large) as the perimeter columns, the interior columns will take roughly twice as much of the lateral load as the perimeter columns. For a symmetrical case, the distribution is shown schematically in Fig. 3-7. The frame of Fig. 3-7 could be either a one story frame or the lowest story of a multistory frame.

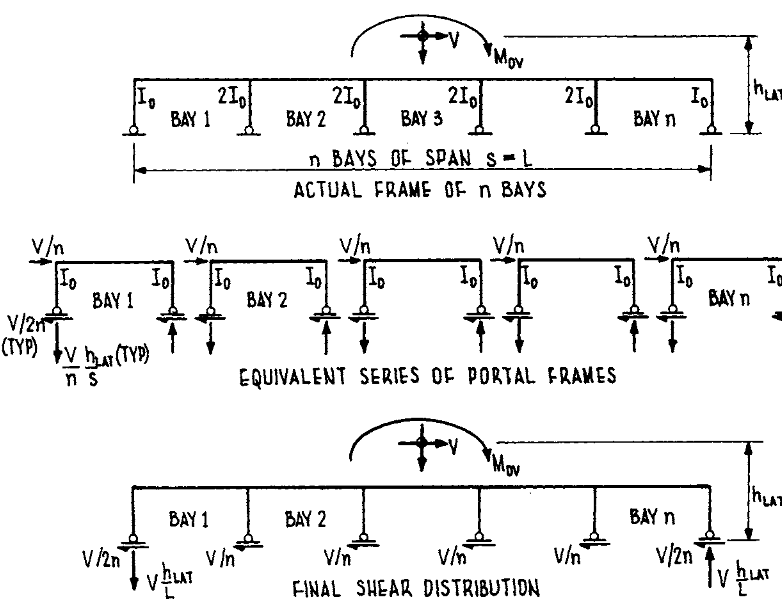


Figure 3-7 Portal Method of Shear Distribution

The distribution of load within a continuous frame is quite simple in a portal analysis. In the frame of Fig. 3-7, for example, there are four interior columns and two perimeter columns, with the interior columns being roughly twice the size of the perimeter columns. The original frame is imagined to be divided into five identical portal frames as shown, with each portal frame representing one bay of the frame. Each portal frame, or bay, then takes one-fifth of the total lateral load.

The overturning load on each portal frame is also included in Fig. 3-7. The restoring moment on each portal frame is shown on the two footings as a vertical couple. The magnitude of this induced vertical force can be found simply by summing moments about either footing of the portal frame.

The final approximate loads on the original frame are found by recombining the *n* portal frames back into the continuous frame, also shown in Fig. 3-7. At each interior footing, the sum of the two vertical loads is essentially zero. The only vertical loads remaining are the couple at the two end footings as shown. Since $ns = L$, the change in load on each end footing is $\pm Vh_{LAT}/L$ vertically and $\pm V/2n$ laterally.

Restoring Moment and Frictional Shear Resistance

Restoring moment and frictional shear resistance are not used for design. They are used only to check for stability of a frame line against sliding or overturning.

The restoring moment on a rigid frame occurs as a reaction to the overturning loads. Statically, it is equal and opposite to the overturning moment. As indicated in Fig. 3-8, the restoring moment M_R is a couple composed of two forces $\pm Vh_{LAT}/L$ located at the end footings, a distance L apart. The restoring moment is thus $M_R = M_{ov} = Vh_{LAT}$

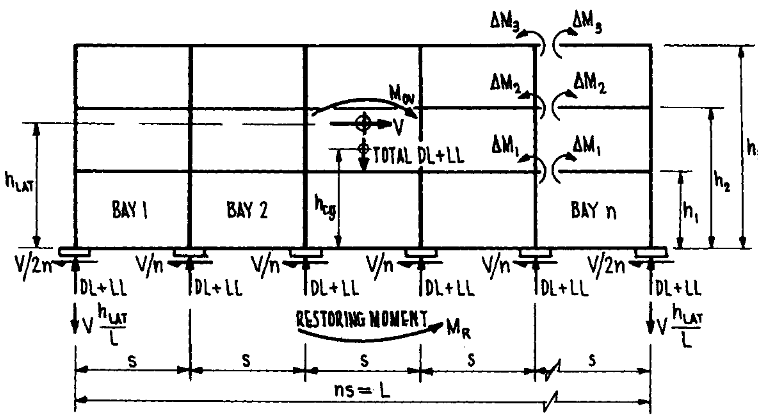


Figure 3-8 Typical Rigid Frame Footing Loads

As indicated, the restoring moment causes added bending in the girders in the two end bays at stories 1 through *m*:

$$M_R = Vh_{LAT} = 2\sum_{i=1}^{m} \Delta M_i \qquad (3\text{-}10)$$

The columns and girders at each level must be designed to sustain the additional moments at that level. (Note that these moments are statically indeterminate.)

The remaining problem to be considered here is the possibility that the footing will slide laterally due to the base shear. Sliding of a rigid frame footing is not usually a problem but it is always a wise precaution to check the maximum friction load that the footing can develop. The possibility of sliding increases as the building becomes lighter and the lateral loads become comparatively larger.

The maximum frictional resistance that the footing can develop is computed as the vertical load times the coefficient of friction μ. The vertical load to be used to determine frictional resistance is specified by Code as the dead load (DL) tributary to the footing. The coefficient of friction between the footing and the supporting soil is prescribed by Code, normally about 0.3.

Drift in a Rigid Fame

A further design limitation in rigid frames is a limitation on the lateral deflection of the frames, called "drift". A sketch of the angle of drift in a rigid frame is shown in Fig. 3-9. At working levels of stress, a nominal maximum limit for the drift angle is 0.005 radians though codes[21,35] permit up to four times this amount in some circumstances.

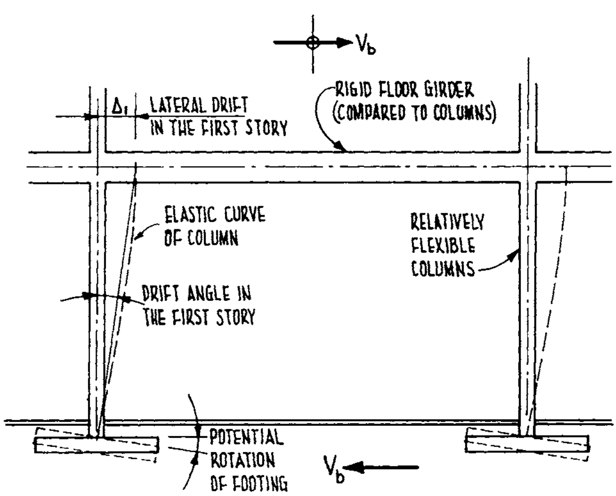


Figure 3-9 Drift in a Rigid Frame

Excessive drift can be highly damaging to glass and finishes as well as to the structural members. The nominal drift angle of 0.005 radians is roughly 1 inch laterally in 16 feet vertically, or about $^5/_8$ inch per story. For rigid frames, drift is controlled by making the columns and girders more flexible or less flexible.

Summary of Foundation Loads in a Rigid Frame

In summary, two completely independent calculations are used to find the applied load and resulting reactions:

- Base shear and overturning moment as applied loads
- Frictional resistance and restoring moment as reactions

Allowable soil pressure p_a' for footings in rigid frames:

A maximum allowable pressure p_a' on the soil is used whenever the soil is subjected to a normal stress combined with a lateral shear stress.

An increase of 33% in p_a' is permitted for live loads and lateral loads when maximum lateral load occurs.

Load cases for combined loads on a rigid frame

The combined load case to be used with the allowable soil pressure p_a' is given by:

$$\text{DL} + 0.75(\text{LL} + \text{W}) \quad \text{or} \quad \text{DL} + 0.75(\text{LL} + \text{E}/1.4)$$

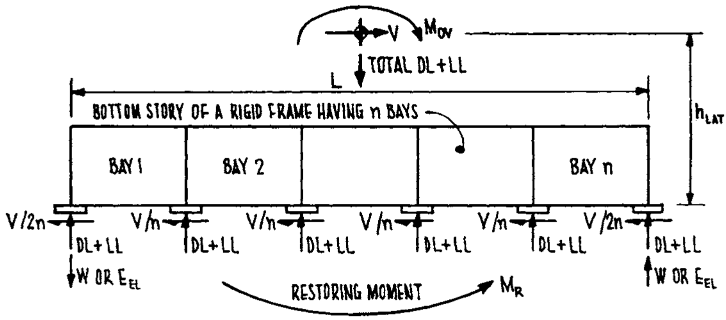
where W and E/1.4 are the vertical loads on a footing (if any) induced by wind or earthquake.

Vertical loads on footings in a frame line

For all footings in a frame line of a rigid frame, the maximum gravity load (DL + LL) on a footing is taken as the sum of all tributary gravity loads halfway to the next vertical support in any direction. The resulting contact pressure on the soil may not exceed the allowable bearing pressure for gravity loads, p_a. (See Chapter 2)

For the interior footings in a frame line, there is no appreciable increase or decrease in vertical load when lateral loads occur. The vertical load on these interior footings is DL + 0.75(LL + 0), to be used with an allowable soil pressure of p_a'. A nominal overstress is allowed for the transient loads.

For the end footings in a rigid frame line, the total combined load on a footing is given either by load case DL + 0.75(LL + W) or by DL + 0.75(LL + E/1.4), where W and E/1.4 are computed as $\pm Vh_{LAT}/L$.



Lateral loads on footings in a rigid frame line

In a frame line of n bays subject to a base shear V:

Every interior footing is subject to an added lateral shear force V/n;

The two end footings in a frame line are subject to an added lateral shear force $V/2n$.

Frictional resistance and restoring moment

Restoring moment and frictional shear resistance are used only to check for stability of a frame line against sliding or overturning.

Frictional resistance of any footing in a frame line is computed as μDL. (μ is coefficient of friction)

Restoring moment is a reaction to the lateral loads; it is equal and opposite to the overturning moment.

Restoring moment is produced entirely by a vertical couple located at the two end footings in a frame line. The magnitude of each couple force is computed as Vh_{LAT}/L.

Restoring moment is unaffected by gravity loads.

Restoring moment produced by the foundation is limited to $M_R = Vh_{LAT}$. The foundation cannot provide any greater restoring moment than this.

Drift limitations for the structure:

The drift of a rigid frame under lateral loads should not exceed a nominal angle of 0.005 radians.

Drift is controlled by varying the configuration or the stiffness of the structure above the footings.

Effect of Lateral Load on Foundations of Braced Frames

For diaphragm-shearwall structures or for braced frames, the distribution of base shear is quite different from that in rigid frames. A typical diaphragm-shearwall structural system is shown in Fig. 3-10, with the forces that resist the lateral load shown at the base of each resisting shearwall.

It should be noted that insofar as the foundation is concerned, it makes no difference whether the "shearwall" is a solid concrete wall, a plywood-sheathed panel, a masonry panel or a cross-braced steel frame. The load delivered to the foundation is the same for any of these rigid panels. In subsequent discussions, it should be understood that the terms "shearwall", "shear panel", "panel" and "braced panel" are often used interchangeably.

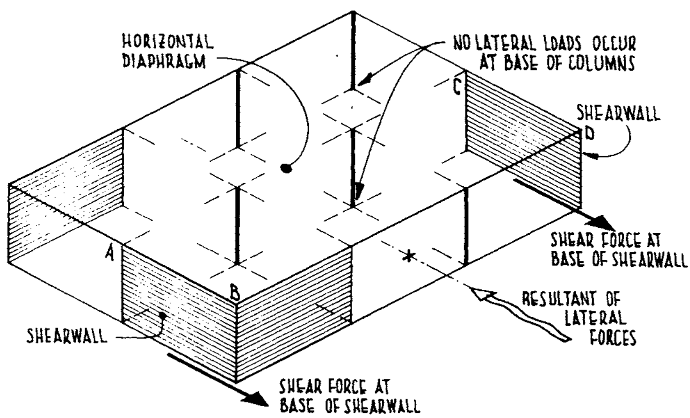


Figure 3-10 Loads on a Diaphragm-shearwall Structure

In Fig. 3-10, the lateral load is resisted entirely by the two panels parallel to the load, shown as panels AB and CD. For the symmetrical case shown, the lateral load is distributed evenly to the two panels, half being resisted by each panel. It is assumed in a diaphragm-shearwall system that all columns are hinged at top and bottom. It is also assumed that shear panels are hinged in their lateral direction at top and bottom, somewhat similar to a giant piano hinge.

Drift in a Braced Frame

Drift is a more important consideration in braced frames than in rigid frames. It is the drift in the shearwall that creates the restoring couple, as indicated in the sketch of Fig. 3-11.

The nominal maximum drift angle for a braced frame is the same as for a rigid frame, 0.005 radians. The control over the drift angle must be exercised by the structural designer when the superstructure is being designed. For the sake of the

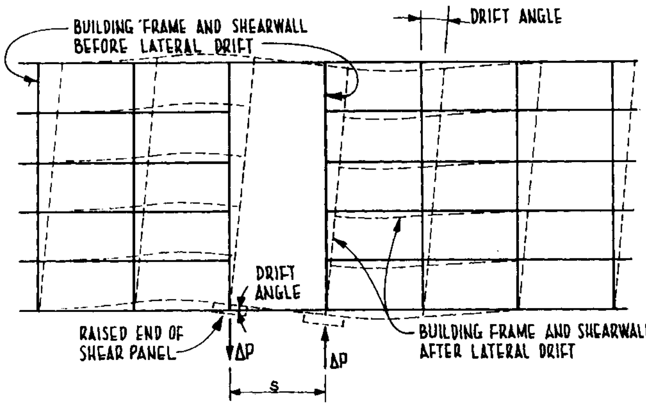


Figure 3-11 Drift in a Braced Frame

foundation design, it will be assumed here that such control of drift has in fact been accomplished in the superstructure and that the drift angle is never more than 0.005 radians, even when lateral load is maximum.

Restoring Moment and Frictional Shear Resistance

Restoring moment and frictional shear resistance are not used for design. They are used only to check for stability of a frame line against sliding or overturning.

A shearwall and its foundations are shown in Fig. 3-12 for two possible configurations. In the first configuration, the shearwall is assumed to carry only shear, with all vertical loads being carried by the columns at each end. In the second configuration, the shearwall is assumed to be a bearing wall as well, having a continuous strip footing.

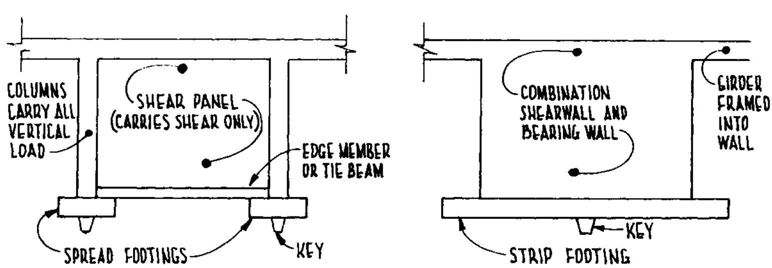


Figure 3-12 Types of Shearwalls and Foundations

As in rigid frames, additional vertical loads are also induced on a braced frame when lateral loads occur. Under the overturning moment M_{ov}, the shear panel will undergo a rotation as shown in Fig. 3-13. The entire building will rotate through a drift angle as indicated, producing a couple in the soil below. The couple is shown as the two forces ΔP at each side of the panel, with $\Delta P = M_{OV}/s$. The uplift side does not create tension on the soil; it is opposed by some portion of the gravity loads, thus reducing the compression at that side.

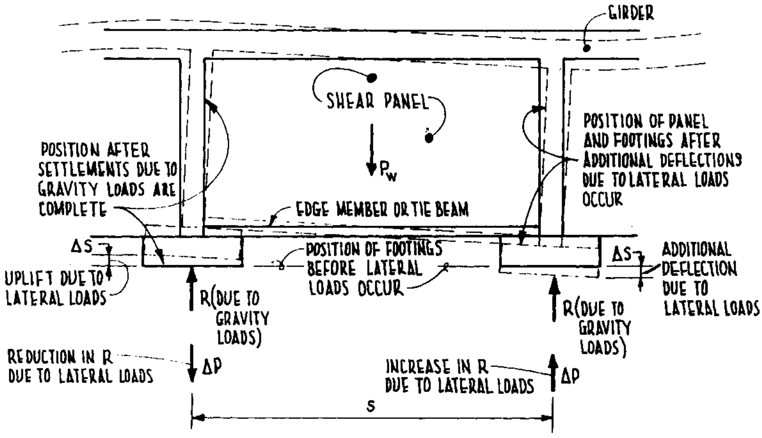


Figure 3-13 Drift Angle in a Diaphragm-Shearwall Structure

The amount of rotation of the footings under this load condition cannot be allowed to exceed the limiting case of 0.005 radians. The rigidity of the structure itself must be designed such that no rotations more than 0.005 radians will occur under this load case.

As the angle of drift increases, the panel of Fig. 3-13 will rotate until at some point the uplift at one end will relieve all of the pressure at that end. At the instant that the footing at one end has just lifted off the soil, the load on the footing at the other end will be the sum of the two original footing loads, or P_W. At this point, the maximum restoring moment has been attained. Further rotation will simply increase the drift angle without increasing the resistance.

At this limiting rotation, the restoring moment of the shearwall footings is then the moment of the couple shown in Fig. 3-13. The couple is composed of the wall load P_W downward and the footing load P_W upward. For this limiting case, the restoring moment M_R of the foundation is the moment of the resulting couple:

$$M_R = P_W \frac{s}{2} \tag{3-11}$$

where P_W is the sum of the dead loads on the two footings at the ends of the panel
s is the span between the two footings

Limiting the load P_w in equation (3-11) only to dead load warrants comment. The worst-case restoring moment is obviously least when the panel load P_w is least; the least load tributary to the panel is therefore taken to be dead load only.

The end conclusion to be drawn is that either of the two footings at the ends of a shear panel can be subject to the combined dead load of both footings. (Remember that lateral load can come from either direction.) The design of these two footings must therefore include this significantly higher vertical load that can occur under lateral loads.

The limitation on the amount of restoring moment that can be developed by shear panels founded on shallow footings is a serious drawback to their use. Short shear panels (less than about 16 feet long) can produce excessively high soil pressures when loaded laterally and may also contribute to excessive amounts of drift. Heavily loaded shear panels are at their best when they can be founded on rigid foundations that can take uplift forces; pile foundations provide such a capability but deep foundations are rarely economical for small structures.

If it should be necessary to increase the restoring moment, either a longer shear panel might be used or an additional number of shear panels might be used. Another possibility is to extend a heavy grade beam outward from the base of the panel to the adjacent footings at either side, as shown in Fig. 3-14. The restoring moment of the panel in Fig. 3-14 is some three times as much as the panel without the extensions.

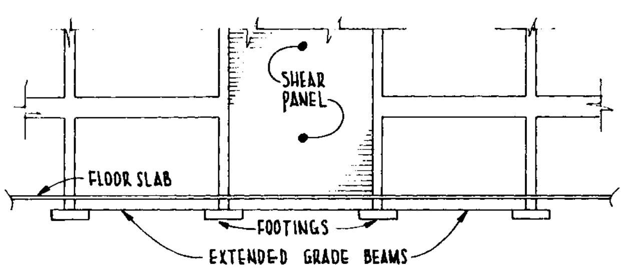


Figure 3-14 Extended Grade Beam

If the foundation supporting a shearwall is a continuous strip footing rather than two spread footings, as shown in Fig. 3-15, the same basic limitation still applies: the shearwall may rotate until the soil pressure at one end of the strip footing is zero, as shown. For the rotated position, the soil pressure diagram for the strip footing becomes triangular and the maximum soil pressure is again roughly double that for the original unrotated position.

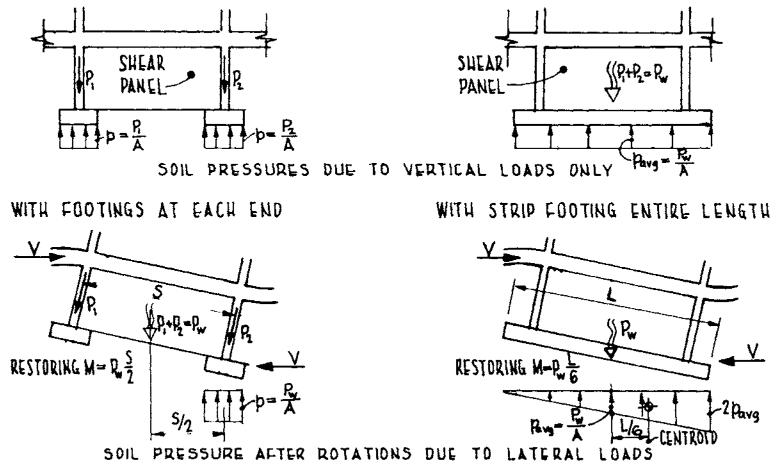


Figure 3-15 Comparison of Shear Panel Footing Pressures

The resultant vertical force for the triangular pressure diagram is at the third point of the triangle. The restoring moment is again the couple formed by the wall load and the resultant of the foundation loads:

$$M_R = P_w \frac{L_f}{6} \tag{3-12}$$

where P_w is the total dead load on the wall
L_f is the total length of the strip footing

A comparison of Equation (3-12) with Equation (3-11) reveals that two spread footings provide some 3 times as much restoring moment as a strip footing of the same general length.

Allowable Soil Pressures for a Braced Frame

The allowable soil pressure p_a' to be used in the design of footings for a laterally loaded braced frame is the same as that described earlier for a rigid frame. As in rigid frames, an increase of 33% in p_a' is permitted for the live load and lateral load components.

As before, the load cases to be used with the allowable pressure p_a' again include 75% of the live load and lateral load,

DL + 0.75(LL + W) or DL + 0.75(LL + E/1.4).

Summary of Foundation Loads for a Braced Frame

In summary, two completely independent calculations are used to find the applied loads and the resulting reactions on a braced frame:

- Base shear and overturning moment as applied loads
- Frictional resistance and restoring moment as reactions

Allowable soil pressure p_a' for footings in braced frames:

A maximum allowable pressure p_a' on the soil is used whenever the soil is subjected to a normal stress combined with a lateral shear stress.

An increase of 33% in p_a' is permitted for live loads and lateral loads when a maximum lateral load occurs.

Load cases for combined loads on a braced frame

The combined load case to be used with the allowable soil pressure p_a' is given by:

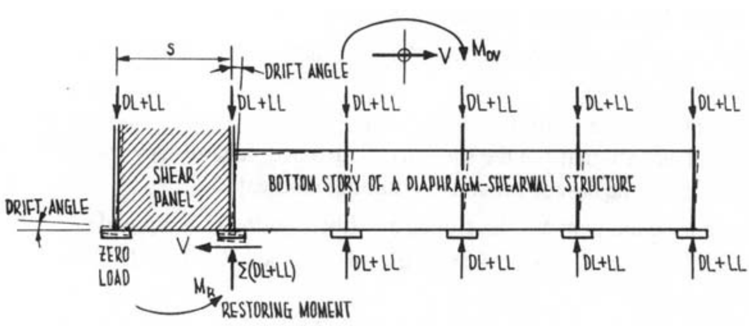
$$\text{DL} + 0.75(\text{LL} \pm \text{W}) \quad \text{or} \quad \text{DL} + 0.75(\text{LL} \pm \text{E}/1.4)$$

where W and E/1.4 are the vertical loads on a footing induced by wind or earthquake, M_{ov}/s.

Vertical loads on footings in a braced frame line

For all footings in a frame line of a braced frame, the maximum gravity load (DL + LL) on a footing is taken as the sum of all tributary gravity loads halfway to the next vertical support in any direction. The resulting contact pressure on the soil may not exceed the allowable bearing pressure for gravity loads, p_a. (See Chapter 2)

On any footing except those supporting a shear panel, there is no appreciable change of vertical load nor is there any base shear introduced when lateral loads occur. The maximum vertical load on these footings remains (DL + LL), to be used with allowable pressure p_a rather than p_a', and with no allowable increase in the bearing pressure p_a.

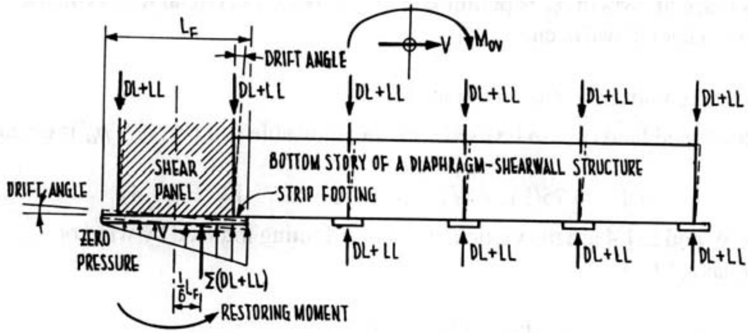


Where two footings are used to support a shear panel undergoing lateral rotations, the soil pressure is assumed to be decreased at the uplifted end of the panel and to be increased at the other end. The load on the footings is computed for an elastic soil:

$$DL + 0.75(LL \pm W) \text{ or } DL + 0.75(LL \pm E/1.4)$$

The maximum values produced by these load cases must be carried by the footings without exceeding the allowable bearing pressure p_a'.

Where a single strip footing is used to support a shear panel undergoing lateral loading, the soil pressure diagram under the footing is taken to be triangular. The pressure is assumed to be zero at the uplifted end of the footing.



The load on the strip footing is computed for an elastic soil:

$$DL + 0.75(LL \pm W) \quad \text{or} \quad DL + 0.75(LL \pm E/1.4)$$

The maximum values produced by these load cases must be carried by the footings without exceeding the allowable bearing pressure p_a'.

Frictional resistance and restoring moment

Two spread footings

Restoring moment and frictional shear resistance are used only to check for stability of a frame line against sliding or overturning.

The entire longitudinal base shear force to be sustained by the shear panel is resisted by friction on the loaded footing only.

The least possible friction force and least possible restoring moment are used to determine stability.

In computing the least friction force and the least overturning moment, only the dead loads tributary to the shear panel are used, regardless what loads were used to determine base shear and overturning moment.

The restoring moment M_R developed by either footing is computed as $M_R = P_w s/2$, where P_w is the sum of all tributary dead loads and s is the footing spacing.

If $M_{ov} > M_R$, the structure is unstable. The additional restoring moment required for stability must be provided elsewhere in the structural design.

Single strip footing

The restoring moment M_R developed by a strip footing of length L_f is computed as $M_R = P_w L_f/6$, where P_w is the sum of all tributary dead loads.

All other design conditions are the same as those given above for two spread footings

Drift limitations for braced frames

The lateral drift of a braced frame or diaphragm-shearwall structure under lateral loads should not exceed a vertical angle of 0.005 radians.

Drift is controlled by varying the configuration or the stiffness of the structure above. Due to the rigidity of a concrete diaphragm-shearwall structure, a drift angle of 0.005 radians can produce serious damage to girders and columns.

One should expect to limit drift in such structures to roughly half the maximum allowable drift of 0.005 radians used for more flexible structures.

The solutions presented here apply to regular modular structures, symmetrical (or roughly so) to the lateral loading. Where the structure is sharply unsymmetrical, the analysis becomes considerably more complex.

Load Combinations for Final Design

It is not reasonable to assume that all of the potential loads that might occur on a structure will occur simultaneously. It is unlikely, for example, that a structure would be subjected to a peak earthquake load while the wind is blowing at maximum velocity. Nor is it likely that the roof live load will be in place during peak wind velocities.

Further, it is not reasonable to require that structural materials sustain transient, rarely-applied loads at the same stress level at which they carry long-term loads. It is the practice, for example, to permit higher stress levels (and possibly even limited damage) over the short duration of an earthquake. Such a practice is made in recognition that the maximum earthquake load will probably occur only once in the service life of the structure.

In addition, it should be recognized that a lesser load rather than a higher load could, in some circumstances, cause "worst-case" stresses. For example, the restoring moment exerted by a shearwall is at its lowest value when the vertical load is minimum, that is, when the vertical load consists of dead load only. For such a case, the existence of live load would serve to increase the restoring moment.

In view of such possibilities, the design of a structure is usually performed for several combinations of load, chosen to produce worst-case stresses under various service conditions. Also, stress levels are commonly increasd by 33% when transient loads such as wind or earthquake are added to the dead and live loads. Further, a check is made for load reversals due to wind or earthquake when vertical load is minimum (dead load only).

For foundation design at working levels of stress in the soil, the following list of load cases are among the more commonly used ones. Special load cases could, of course, be required for unusual circumstances. It was observed earlier that using 75% of a load at the usual allowable stress will produce a 33% overstress whenever 100% of the load occurs.

Dead load + 100% live load	(3-13a)
Dead load + 50% live load	(3-13b)
Dead load + 75%(live load + wind)	(3-13c)
Dead load + 75%(live load + earthquake/1.4)	(3-13d)
Dead load + wind load	(3-13e)
Dead load + earthquake/1.4	(3-13f)

It should be recognized in the foregoing load combinations that four of the combinations include lateral loads, for which the allowable soil pressure is denoted p_a'. The remaining two load combinations are used to design for two conditions, the first for strength and the second for settlement. The allowable soil pressure for strength is denoted p_a and for settlement p_a''.

As an observation, it seems likely that the maximum wind or earthquake load will occur under day-to-day circumstances when the live load is somewhat less than 100%. However, the 33% overstress is allowed only with 100%LL which, in essence, encourages a designer to use the higher load case.

It is also observed that the 1997 UBC Section 1630.1 prescribes an amplification factor W_0 to be applied to the earthquake forces computed from Equation (3-5). Other modifications to the earthquake forces are also prescribed to improve reliability and redundancy. These modifications are most applicable in strength design; since foundations are designed elastically, their application here is limited.

Applications in Determination of Design Loads

The final design loads on footings for the building shown in Chapter 2, Fig. 2-1, can now be determined. The foregoing concepts and procedures will be used to find the loads on footings at grid points B3 and A2, considered to be typical shallow foundations

Example 3-3 Final design loads on footings

Given : Building and conditions given in Chapter 2, Fig. 2-1

To find: Design loads for the footing at grid point B3 for the following cases:

1. Dead Load plus Live load
2. Dead load plus 50% live load
3. Dead load plus live load plus wind load
4. Dead load plus live load plus earthquake

Solution:

Since footing B3 is one of the interior footings in a diaphragm-shearwall system, it is not subject to lateral load.

The dead load and live load on the footing at grid point B3 are computed in Examples 2-1 and 2-2. The loads are:

At footing B3, $P_{DL} = 214$ kips, $P_{LL} = 114$ kips

For strength considerations, the load combination to be used is DL + LL with an allowable soil pressure p_a. For settlement considerations, the load combination is DL + 50% LL with an allowable soil pressure p_a''

For footing B3, DL+LL = 328 kips; DL+50% LL = 271 kips

For footing B3, load combinations 3 and 4 do not apply

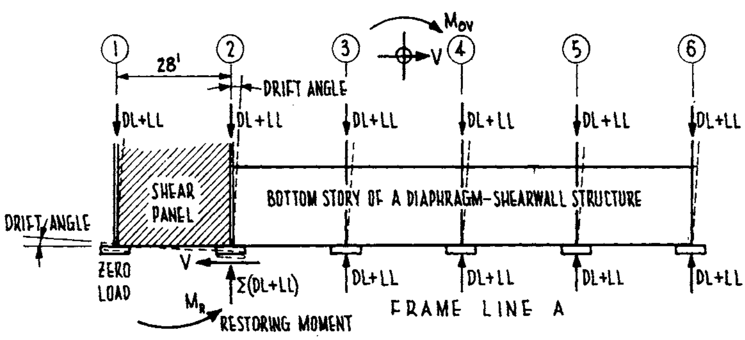
Example 3-4 Final design loads on footings

Given : Building and conditions given in Chapter 2 preceding Fig. 2-1.

Base shear V, overturning moment M_{ov} and height to center of lateral forces h_{LAT} are determined in Examples 3-1 and 3-2:

For wind, $V = 72$ kips (two panels)
$M_{ov} = 1640$ kip•ft(two panels)
$h_{LAT} = 22.8$ ft

For earthquake, $V = 301$ kips (two panels)
$M_{ov} = 8700$ kip•ft(two panels)
$h_{LAT} = 28.9$ ft.



To find: Design loads for the footing at grid point A for the following load cases:

1. Dead Load plus Live load
2. Dead load plus 50% live load
3. Dead load plus live load plus wind load
4. Dead load plus live load plus earthquake

Solution:

It is noted that footing A2 is one of the footings at the end of a shearwall. The loads on footing A2 cannot therefore be computed independently; the loads on footing A1 at the other end of the shearwall must also be included.

The loads on footings A1 and A2 have been computed aside, following the procedures used in Examples 2-1 and 2-2. The loads are:

At footing A1, $P_{DL} = 183$ kips, $P_{LL} = 24$ kips
At footing A2, $P_{DL} = 219$ kips, $P_{LL} = 49$ kips

When no lateral loads occur, the load combination to be used for strength considerations is DL + LL with an allowable soil pressure p_a.

For footing A-1, DL + LL = 207 kips
For footing A-2, DL + LL = 268 kips

For settlement considerations, the load combination is DL + 50% LL with allowable soil pressure p_a''

For footing A-1, DL + 50% LL = 195 kips
For footing A-2, DL + 50% LL = 244 kips

For lateral loading, the shearwall footing sizes are designed at maximum combined load without exceeding the allowable soil pressure p_a'. The vertical loads induced on the panel footings by the overturning moment on one shear panel are computed as M_{ov}/s where s is the spacing of the footings, 28 ft.

For wind, W = (1640/2)/28 = 29 kips
For earthquake, $E_{ELASTIC}$ = (8700/2)/28 = 155 kips

The design loads for footings A1 and A2 is the sum of all combined loads tributary to the panel.

For footing A1:

for wind,
$$P_w = DL + 0.75(LL \pm W) = 183 + 0.75(24 \pm 29) = 179 \text{ kips or } 223 \text{ kips}$$

for earthquake,
$$P_w = DL + 0.75(LL \pm E_{ELASTIC}) = 183 + 0.75(24 \pm 155) = 85 \text{ kips or } 317 \text{ kips}$$

For footing A2:

for wind,
$$P_w = DL + 0.75(LL \pm W) = 219 + 0.75(49 \pm 29) = 234 \text{ kips or } 278 \text{ kips}$$

for earthquake, $P_w = \text{DL} + 0.75(\text{LL} \pm \text{E}_{\text{ELASTIC}})$
$= 219 + 0.75(49 \pm 155)$
$= 140$ kips or 372 kips

It is noted that under elastic response, no uplift will occur at either footing under maximum lateral load.

The foregoing design loads are summarized in the following list for future reference.

Design loads for footing A1

$V = 36$ kips and $M_{ov} = 820$ kip•ft (wind)
$V = 155$ kips and $M_{ov} = 4350$ kip•ft (earthquake)

$P_{DL} = 183$ kips $P_{LL} = 24$ kips
$W = 29$ kips $E_{ELASTIC} = 155$ kips

Maximum gravity load = 207 kips
Maximum sustained load = 195 kips
Maximum combined load = 223 kips (wind)
Maximum combined load = 317 kips (earthquake)

Design loads for footing A2

$V = 36$ kips and $M_{ov} = 820$ kip•ft (wind)
$V = 155$ kips and $M_{ov} = 4350$ kip•ft (earthquake)

$P_{DL} = 219$ kips $P_{LL} = 49$ kips
$W = 29$ kips $E_{ELASTIC} = 155$ kips

Maximum gravity load = 268 kips
Maximum sustained load = 244 kips
Maximum combined load = 278 kips (wind)
Maximum combined load = 372 kips (earthquake)

As a matter of interest, the overturning moment M_{ov} on one panel is 4350 kip•ft. The restoring moment M_R is computed as

$$P_w s/2 = (207 + 268)(28/2) = 6650 \text{ kip•ft for one panel,}$$

which is larger than the overturning moment. The structure is thus stable for overturning.

Review Questions

3.1 How are the effects of wind and earthquake similar in the way they act on a building?

3.2 How does wind create base shear on a structure?

3.3 How does earthquake create base shear on a structure?

3.4 Why don't wind and earthquake produce differential settlements between footings?

3.5 How does a lateral load against a building produce additional vertical loads on the footings?

3.6 Why is an overstress permitted when maximum lateral load occurs in combination with gravity loads?

3.7 How much overstress is permitted when maximum lateral load occurs in combination with gravity loads?

3.8 How is the allowable overstress due to lateral loads incorporated into the design of the foundations?

3.9 How is the base shear due to wind or earthquake resisted by the foundation?

3.10 What is meant by "wind stagnation pressure"? How is it computed?

3.11 Why is it necessary to determine the wind stagnation pressure at some reference elevation above the ground?

3.12 How is stagnation pressure converted into the pressure actually acting against a building?

3.13 How is the variation in pressure along the height of a structure determined?

3.14 What is meant by the "silhouette" of a structure?

3.15 How much is the wind pressure against a structure at ground level?

3.16 What is meant by "overturning moment" due to wind load? How is the overturning moment determined?

3.17 Define the "importance factor" in the design of buildings for wind load. To what kind of buildings might it apply?

3.18 Where does one find the value of the "importance factor" that is to be used in the design of a project?

3.19 How is earthquake base shear computed for low buildings?

3.20 Where does one find the value of the seismic zone factor to be used for a particular project?

3.21 How is live load treated in an earthquake analysis?

3.22 How is the base shear distributed to the individual footings along a rigid frame line?

3.23 How is the overturning moment resisted by a rigid frame line?

3.24 At the interior footings in a rigid frame line, there is no resultant vertical load due to wind or earthquake. Why not?

3.25 At all footings in a braced frame except those supporting the shear panels, there is neither vertical nor lateral load introduced due to wind or earthquake. How is that possible?

3.26 What is the interactive response factor R used in earthquake design?

3.27 What is meant by "g-load" due to earthquake?

3.28 How is base shear resisted in a braced frame line?

3.29 How is "drift" controlled in a rigid frame line?

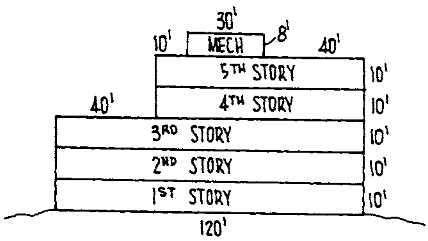
3.30 How is "drift" controlled in a braced frame line?

3.31 What is the maximum restoring moment that can be developed by the shear panel if the foundation consists of two spread footings? A strip footing?

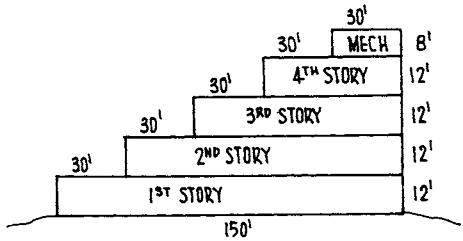
OUTSIDE PROBLEMS

Problems 3.1 through 3.6. Determine the wind pressures against the building silhouettes shown. Find the loads acting at each story, the overall base shear, the overturning moment and the height of the center of lateral wind forces, h_{LAT}.

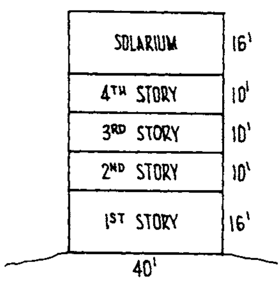
3.1 Wind velocity 100 mph



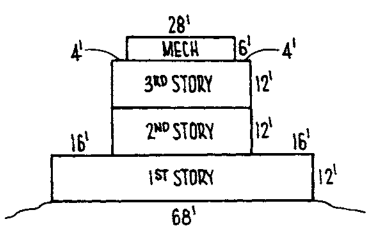
3.2 Wind velocity 90 mph



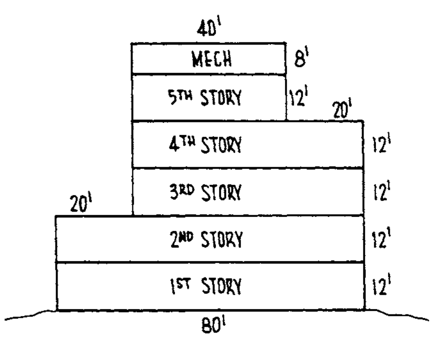
3.3 Wind velocity 70 mph



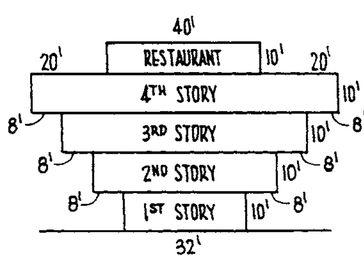
3.4 Wind velocity 90 mph



3.5 Wind velocity 80 mph

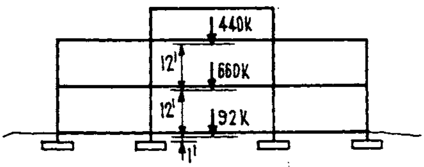


3.6 Wind velocity 110 mph

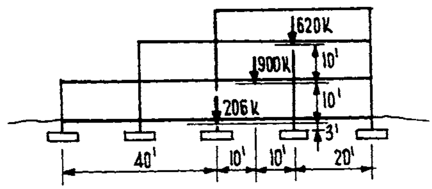


Problems 3.7 through 3.12. Determine the earthquake forces acting on the indicated frames. Dead loads are given at each floor level, assumed to fall at the bottom face of the floor slab. Find the c.g. of dead loads, the lateral inertia load at each story, the overall base shear, the overturning moment and the height of the center of lateral inertia forces, h_{LAT}.

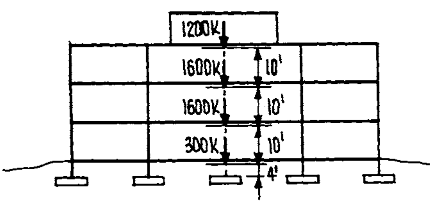
3.7 Seismic risk zone 3
Soil profile type S_D
Steel rigid frame



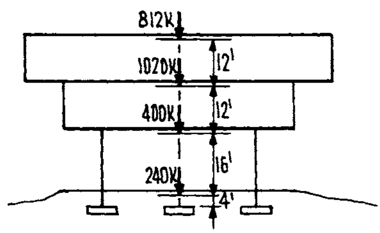
3.8 Seismic risk zone 2A
Soil profile type S_E
Concrete rigid frame



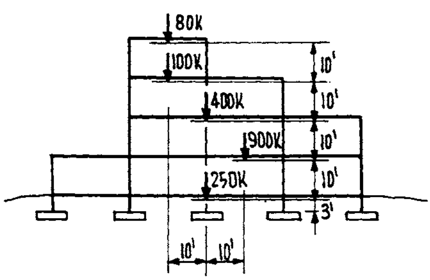
3.9 Seismic risk zone 2B
Soil profile type S_E
Concrete diaphragm-shearwall



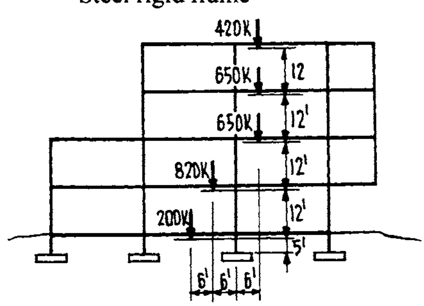
3.10 Seismic risk zone 3
Soil profile type S_D
Steel braced frame



3.11 Seismic risk zone 3
Soil profile type S_D
Concrete diaphragm-shearwall



3.12 Seismic risk zone 3
Soil profile type S_E
Steel rigid frame



Chapter 4

CLASSIFICATIONS AND PROPERTIES OF SOILS*

Broad Soil Groupings

In contemporary construction, soil is simply another construction material. The soil, however, is acquired along with the building site and in most cases must be used "as is". Engineers do not have the latitude to specify the engineering properties of soil in the way they do for steel, concrete or timber.

As a construction material, soil is especially notable for its variability. Even within a single building site, there can be wide variations in soil properties. A detailed study of the various types of soil and their engineering properties is a first step in learning to accommodate such variability. Such a study of soil types is introduced in this chapter.

Soil is classified into four broad groupings: gravels, sands, silts and clays. These four classifications are developed in considerable detail in this chapter.

Gravels and coarse sands respond to foundation loads in essentially the same way. There is therefore no need to discuss them separately as foundation materials. Subsequent discussions of sands as a foundation material should be understood to include gravels also.

Sands are familiar to everyone who has ever been to the beach or desert. Generically, sands are called "cohesionless" soils to describe their total lack of tensile strength. The tendency of sand particles to cling together when wet is only an apparent cohesion; it is not a permanent tensile strength. All of the strength of sand is derived from intergranular friction and intergranular interlock.

Clays are familiar to anyone who has ever seen or used modeling clay. Its sticky, almost greasy texture is evidence of its tensile strength. All the strength of clay comes from its tensile strength. Clays are generically called "cohesive" soils.

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* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

Silts are less distinctive than either sands or clays and are not as readily identified. Some silts are cohesive, some are cohesionless. In appearance, silt looks like a very fine dusty powder.

Organic soils may be prized for agricultural production, but they are not suitable for carrying foundation loads. In a building site, organic soils will probably have to be removed from the foundation area and replaced by inorganic soils. No further consideration of organic soil as an engineering material is included here.

The foregoing four broad groupings can be collected into two major soil types according to the way they respond to foundation loads.

The first major soil type consists of the cohesionless soils: gravels, sands and cohesionless silts. In all succeeding discussions, a general reference to sands is meant to include gravels and cohesionless silts as well as sands. These three soils respond to loads in the same general way.

The second major soil type consists of the cohesive soils: clays and cohesive silts. As before, a general reference to clays is meant to include cohesive silts as well as clays. These two soils respond to loads in the same general way.

The foregoing breakdown of soils into sands and clays is made in recognition of the way the two types of soil respond to load. While a convenient distinction, it should be noted at this point that there is a vast difference between the way sands respond to load and the way clays respond to load.

Response of a Soil to Foundation Loads

Sands and clays are thus the two extremes in soil types that may be encountered in foundation design. Most soils, however, are neither pure sand nor pure clay, but are random mixtures of these extremely different types of soil. It will be found in later dicussions, however, that the response to load by such mixtures can still be predicted with a reasonable degree of confidence.

A comparison of ways in which a "pure" sand and a "pure" clay respond to load is shown in Table 4-1. In the table, item 1 for footings on sand compares to item 1 for footings on clays; other item numbers correspond similarly. At the two extremes shown in Table 4-1, it should be noted immediately that the two soils often respond to load in diametrically opposite ways.

Table 4-1 is developed fully in Chapters 6 and 8. It is moved forward into these discussions only to point out the sharp differences in the way the two types of soil respond to load. Each item in the table is verified in Chapters 6 and 8.

It should be evident from the comparisons of Table 4-1 that it is essential to identify the foundation material before beginning the structural design. Proceeding on assumptions or on past practices could require expensive (and embarrassing) redesign at a later date.

Table 4-1 COMPARISON OF RESPONSE TO LOAD

Footings on Sands	Footings on Clays
1. Sand strength increases with confining pressure, whether due to overburden or to footing loads.	1. Clay strength is relatively constant, regardless of the magnitude of any confining pressure.
2. Sand strength is due entirely to friction, measured by the ngle of internal friction ϕ.	2. Clay strength is due entirely to cohesion, or tensile strength
3. Sand strength is relatively insensitive to footing shape.	3. Clay strength is influenced considerably (up to 20%) by footing shape.
4. Sand strength increases markedly (doubled or tripled) with depth of burial of the footing.	4. Clay strength is relatively insensitive to depth of burial of footing.
5. Loss of strength of sand is significant if the overburden confining pressure) is removed or is eroded away.	5. Little loss of strength of clay is caused by removal of overburden.
6. Strength of a dry sand is cut in half when the sand is submerged in water.	6. Strength of clays is relatively unaffected by short-term submergence.
7. Settlements in sands occur soon after application of load, measured in weeks or a few months.	7. Settlements in clays occur *very* slowly following application of load, measured in months or years.
8. Settlements in sands can occur under relatively short-term loads.	8. Settlements in clays are relatively unaffected by short-term loads.
9. Settlements in a dry sand are essentially doubled if the sand is submerged (or if the water table rises).	9. Settlements in clays are markedly affected (but not doubled) if the clay is submerged.
10. Deposits of sand are best compacted by vibration and submergence, with some pressure.	10. Deposits of clay are best compacted by long-term surcharge pressure.

Geologic Origins of Soil

It is a feature of earth's geologic activity that the rock forming the earth's mantle is constantly being thrust upward out of the mantle onto the surface. Once exposed, the rock is subject to earthquakes, glaciation, freeze-thaw, water erosion, chemical attack, waterborne abrasion, wind erosion and other forms of weathering. Under such relentless attack over millions of years, the mountainous rocks are broken progressively into huge fragments, these fragments into huge boulders, these into smaller boulders, these into cobbles, these into pebbles and finally, the pebbles are reduced to grains. Deposits of these assorted particles of rock and rock grains are called *soils*.

The word in engineering geology that is used specifically to describe the ever-continuing reduction of rock into a smaller and smaller gradation is *fragmentation*. Fragmentation of rock is not something that happened in some vague geologic age. It is a continuing process, as active today as it was a hundred million years ago.

As the size of the particles becomes progressively smaller, the particles become progressively easier to transport. Transported at first by such things as glaciation and avalanche, the particles are reduced to smaller and smaller sizes as they are subjected successively to rushing floods, then when smaller, by whitewater rapids, later by rapidly moving streams, eventually by muddy meandering rivers, and finally, when small enough, to wind and duststorms. At any stage, the particles might be deposited in a recognizable stratum for a few thousand years before something happens that causes them to be picked up, transported and deposited in some new location along with particles transported similarly from hundreds of other locations.

At every stage of deposition and stratification, water is the ever-present medium of erosion and transport. Water deep within the ground carries acids and bases that chemically attack the particles even when they are buried thousands of feet deep. At the surface, rain, snow and sleet combine to erode, freeze, thaw and further reduce chunks of rock into ever smaller and smaller pieces.

Whenever exposed, the soil particles are subject to organic attack by vegetation, carbon dioxide and atmospheric acids, changing them chemically into other compounds or even into other minerals. Picked up again, redeposited and exposed again and again, the particles might undergo thousands of years of changes before they come to rest for a few hundred years in relative quiet. It is at one of these quiet periods that the foundation engineer is given a stratum of these particles on which to place a foundation.

Because nature works on such a huge scale, such strata are usually (but not always) so large that a foundation can be located entirely on one stratum. But underlying this stratum could be another stratum having vastly different engineering properties and under that, yet another. The material to be used to support a building foundation is thus a heterogeneous mixture of minerals coming from countless sources over the breadth of a continent, randomly deposited and irregularly stratified. It is, in short, soil.

With such a description, defining the engineering properties of a material as variable as soil would seem to be hopeless. In recent times, however, real progress has been made in defining the engineering properties of soils. Although these properties are more appropriate within large brackets rather than in refined details, the response of most soils to a bearing load can now be predicted with some degree of confidence.

Insofar as the engineering properties of a soil are concerned, the mechanism of geologic transport, deposition, burial and exposure is one of the more important influences. With few exceptions, all soils have been transported to their present locations from somewhere else. They have been blended, disturbed, chemically modified, reblended, mixed, crushed, restratified, picked up and redeposited, sometimes loaded by thousands of feet of overburden and finally exposed when the overburden was eroded away. The mechanism by which the stratum was finally deposited and later exposed will be seen to be of profound importance when the soil is used as a foundation material.

A second major influence is the groundwater. The location of the water table, the amount of its rise and fall throughout the year and the chemistry of the groundwater can profoundly affect both the type of foundation and ability of the soil to carry the foundation loads. In some circumstances, however, the water entering the soil from the surface can be far more important than that in the water table below; this point is developed further in later chapters.

A third major influence is the residue of vegetation. Even in the most barren deposits of soil, a few plants will somehow survive. The residue from their eventual death and decay will provide a somewhat more hospitable environment for the next generation of plants. The dead remains, or *detritus* of these plants in turn will deepen the fragile layer of organic material, providing a yet more fertile ground for other plants, and so on. Eventually, a gradient of organic material is developed, with the high organic content at the surface diminishing steadily with depth. As always, water is the primary vehicle for transporting the organic material downward, aided in this case by flow lines left from old root penetrations.

Soil Profiles and Soil Horizons

The combined effects of deposition, water percolation, vegetation and other influences on the soil eventually produce a typical distribution of soil and organic matter called a *soil profile* by agronomists. Such a profile, divided into *soil horizons*, is shown schematically in Fig. 4-1 for a soil in a temperate climate. The typical thicknesses shown in Fig. 4-1 are intended only as a broad indication. The thicknesses of the various horizons would be significantly larger for the rich soils of the valleys and bottomlands and much shallower for the sparse soils of the mountain slopes.

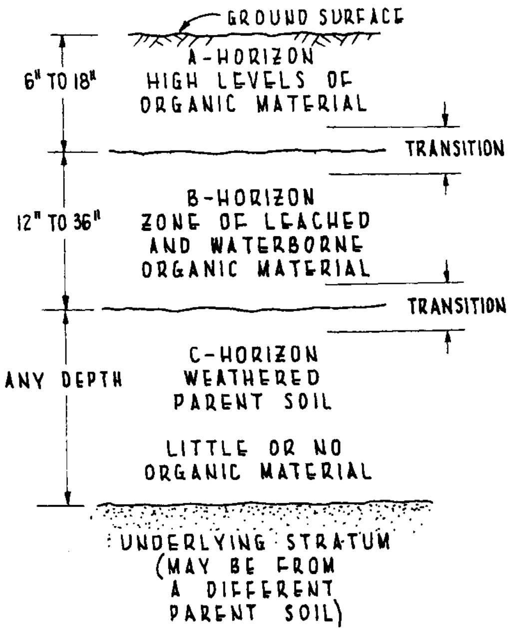


Figure 4-1 Soil Horizons in a Soil Profile

The presence of humus or organic material in a soil can cause serious changes in the engineering properties of the soil as the organic material continues to decay. Due to this variability, engineered foundations are placed in the parent soils of the C horizon, well below the organic agricultural soils of the A and B horizons. The engineering properties of the soils in the C horizon are not affected by further decay of the organic material above. All further discussions in this book are limited to these inorganic (or minimally organic) soils of the C horizon.

Although soils have long been classified into broad groupings, the engineering properties of various classifications of soils cannot be determined once and used thereafter as constants; a soil in Wyoming classified as a clay might have vastly different properties from a soil in Georgia classified as a clay. There are, however, certain characteristics that are common to all soils regardless of origin or chemistry. Such characteristics can be used as a basis for soil classifications. Discussions of the more prominent of such characteristics follow.

Grain Size and Distribution

The size, shape and distribution of the particles that make up a soil have long been used to describe the soil, first by agrarians and later by engineers. Several methods of screening or sieving to determine the sizes of the particles have been developed; all have some degree of merit. The system presented here is the one currently most popular with engineers, that prescribed by the American Society for Testing and Materials under its designation ASTM D422.

A sketch of a typical granular soil structure is shown in Fig. 4-2. The larger particles create void spaces which are filled by smaller particles, the remaining voids are, in turn, filled with smaller particles and so on. When the size and gradation of the soil particles are such that the final amount of void space is minimal and the soil approaches its maximum possible density, the soil is said to be well graded.

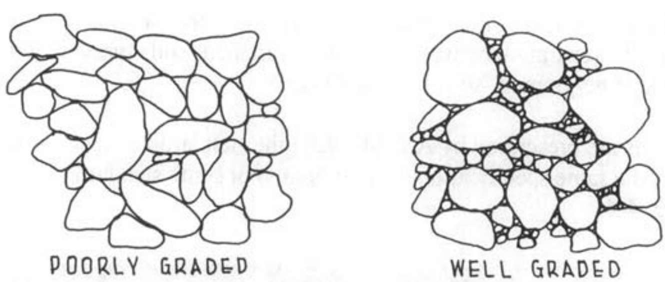


Figure 4-2 Granular Soil Structure

Granular soils have been further subdivided into general groupings according to their grain size. As indicated in Fig. 4-3, for example, a soil composed of particles having sizes between 0.075 mm and 0.425 mm is classified as a fine sand. These particle sizes will pass through a standard sieve having 40 wires per inch but will be retained on a standard sieve having 200 wires per inch.

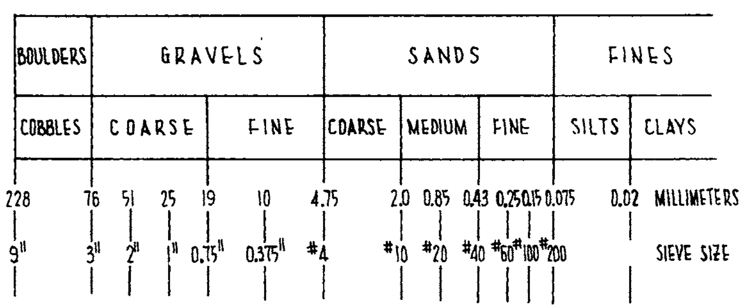


Figure 4-3 Particle Sizes and Sieve Identification

Although many Sieve sizes are available, the more commonly used sizes are those listed in Fig. 4-3. The clear opening between wires is given in inches only down to $^3/_8$ inch. Thereafter, the sieve size is given as the number of wires per inch rather than clear opening.

Natural break points between soil classifications fall at certain sieve sizes. Such break points are indicated in Fig. 4-3. Among the most-used of these break points are the following sieve sizes:

3 inch sieve separating cobbles from gravels

No. 4 sieve separating gravels from coarse sands

No. 40 sieve separating medium sands from fine sands

No. 200 sieve separating coarse-grained soils (gravels and sands) from fine-grained soils (silt and clays).

By far the most significant of the natural break points is the No. 200 sieve size, which defines the separation between the "coarse-grained" soils (gravels and sands) and the "fine-grained" soils (silts and clays).

Standard sieves are prescribed by ASTM D422; the standardized test is also prescribed in the same specification. A photograph of some standard sieves is shown in Fig. 4-4.

Figure 4-4 Nesting Sieves and Powered Shaker
(Photos courtesy ELE / Soiltest)

In a standard sieve analysis prescribed by ASTM D422, a soil sample is first weighed, then passed through a set of standard sieves. The total weight passing the 3 inch sieve is recorded, then successively, the total weight passing the 1 inch, $^1/_2$ inch, No. 4, No. 20 and so on. The results are plotted on semilog paper, producing a *gradation curve* such as one of those shown in Fig. 4-5.

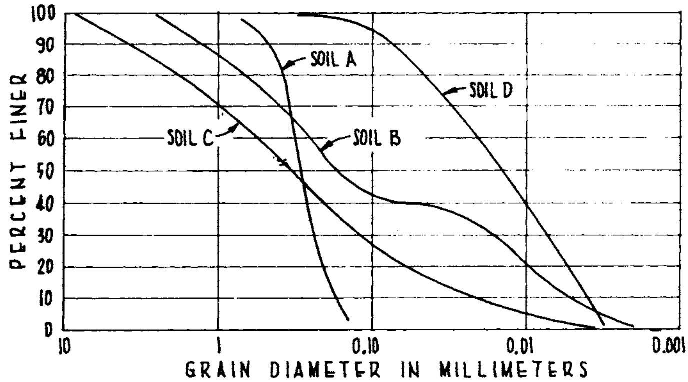


Figure 4-5 Example Gradation Curves

The gradation curves of Fig. 4-5 indicate several distinctive types of coarse-grained soils. Curve C indicates a *well graded* granular soil, having grain sizes well distributed throughout the normal range of grain sizes. The smooth, gentle slope and curvature of curve C indicates that the soil has no excessive amount of particles in one size nor any gaps in another size.

Curve A, in contrast, indicates a *uniformly graded* soil, having most of its particles grouped together in one narrow range of sizes. Beach sands that have had their finer sizes removed by water erosion will typically produce a gradation curve similar to curve A.

Curve B in Fig. 4-5 indicates the gradation of a *gap-graded* soil in which a complete range of sizes is missing. Such soils can come about by marine deposition, where soil particles have been deposited by fast-moving water followed by deposition by slow-moving water, with nothing in between. Sometimes, blending a soil of this type with a uniformly graded soil such as that of curve A will produce a better gradation.

Soils are often described as being *coarse-grained* or *fine-grained.* A coarse-grained soil is defined as one in which more than 50% of the particles are *retained* on the No. 200 sieve. A fine-grained soil is defined as one in which 50% or more of the particles *pass* the No. 200 sieve.

In discussing coarse-grained soils such as those of Curves A, B and C, it is often useful to designate grain sizes at certain points along their gradation curves. As an example, the designation D_{60} is used to denote the sieve size which allows 60% of the material to pass; 60% of the particles are therefore smaller than the D_{60} sieve size. Similarly, 30% of the particles are smaller than the D_{30} sieve size and 10% of the particles are smaller than the D_{10} sieve size. (The D_{10} sieve size is often called the *effective size* of a coarse-grained soil; it is used to define the permeability of the soil.)

The sieve sizes are also used to describe general properties of coarse-grained soils. For example, the *uniformity coefficient*, C_u, is a measure of the uniformity of soil particles and is computed by

$$C_u = \frac{D_{60}}{D_{10}} \tag{4-1}$$

Similarly, the smoothness of the gradation curve is indicated by the *coefficient of curvature* , C_z, and is computed by

$$C_z = \frac{D_{30}^2}{D_{60} D_{10}} \tag{4-2}$$

The coefficients C_u and C_z are empirically developed. They provide a means to determine whether or not a coarse-grained soil is well graded. For gravels, if $C_u > 3$ and $1 < C_z < 3$, the gravel is well graded; for sands, if $C_u > 6$ and $1 < C_z < 3$, the sand is well graded. Any coarse-grained soil not meeting these requirements is considered by default to be poorly graded.

The soil shown by curve D in Fig. 4-5 is described as a fine-grained soil since more than half the soil particles pass the No. 200 sieve. For such fine soils, a gradation curve has little or no meaning. A more definitive property is obviously needed when dealing with soils having extremely small particle sizes. Such a property in fine-grained soils is the *plasticity* of the soil. The property of plasticity in fine-grained soils is presented in the following section.

Plasticity and Atterberg Limits

Fine-grained soils are very sensitive to the amount of water in their void spaces, or *pores*. Fine-grained soils will even change their physical state as their water content increases, going from a solid to a plastic to a liquid. Coarse-grained soils undergo no such change of state.

An example of this phenomenon in fine-grained soils occurs in modeling clay. At the preferred water content, modeling clay is in a plastic state, readily molded into various shapes. As it dries, it becomes solid, even brittle. Adding water to the dried clay will restore its plasticity, but if too much water is added, the clay will pass beyond plasticity into a soupy or liquid state.

Four distinct states —— solid, semisolid, plastic and liquid —— can be distinguished and identified in a fine-grained soil by the amount of porewater it contains. A general description of these states is is given in Table 4-2. While the descriptions in Fig. 4-2 may be easily visualized, they are highly subject to one's personal bias; a more measurable distinction between states is obviously needed.

Table 4-2 PHYSICAL STATES OF FINE-GRAINED SOILS

States	Limits	Comparative Description
Solid		Rock candy
	Shrinkage Limit	
Semisolid		Firm swiss cheese
	Plastic Limit	
Plastic		Modeling clay
	Liquid Limit	
Liquid		Thick pea soup

A fine-grained soil changes from a solid state to a semisolid state at the *shrinkage limit*, from a semisolid state to a plastic state at the *plastic limit* and from a plastic state to a liquid state at the *liquid limit*. As a means to remove the element of personal bias in establishing these limits, standardized tests are now used which are widely accepted in the practice.

The points at which a fine-grained soil changes its physical state are commonly called the *Atterberg Limits*, being named after the soil scientist who introduced them[1]. They are determined for that portion of a soil that passes the No. 40 sieve; some fine sand is therefore included in the standard tests. Only the plastic limit and the liquid limit are presented here. The third Atterberg limit, the shrinkage limit, has fallen out of use in modern practice.

Atterberg limits are stated in terms of the *water content*. The water content of a soil, w, is a ratio of weights. It is defined as the weight of water in the voids, W_w, divided by the weight of the solids, W_s. It is generally stated as whole numbers of percent but without the percent sign.

Water content w is computed as:

$$w = \frac{W_w}{W_s} x100 \qquad (4\text{-}3)$$

A water content of 20 therefore means that the water in the sample weighs 20% as much as the solids. The term "water content" is one of several *index properties* used in describing the engineering properties of soil. Other index properties are introduced later in this chapter.

The plastic limit of a soil, w_P, is the water content at the point when the soil passes from a solid state to a plastic state. The test for the plastic limit (called the rat-tail test) consists of rolling a small sample manually into a string $^1/_8$ inch in diameter. When the water content is at a point such that the string just starts to crumble when it is at $^1/_8$ inch diameter, the soil is at its plastic limit.

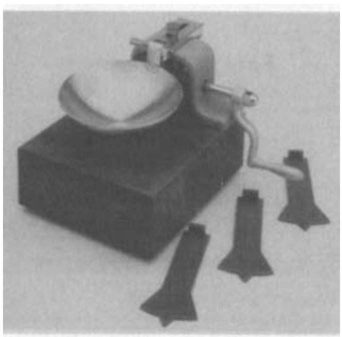


Figure 4-6 Standard Liquid Limit Test Device (Photo courtesy ELE / Soiltest)

The liquid limit of a soil, w_L, is the water content at the point when the soil passes from a plastic state to a liquid state. The test for liquid limit requires a simple mechanical device, shown in Fig. 2-6. The device was expressly developed to remove personal bias or opinion from the execution of the test.

In conducting the test for liquid limit, a sample of moist soil is placed in the cup of the liquid limit device and a groove is formed in the sample with a standard grooving tool. The crank is then turned, causing the cup to rise exactly 10 mm and then fall back onto the base. Each such fall is called a "blow". When the water content is such that a $^1/_2$ inch length of the groove closes at 25 blows, the soil is at its liquid limit.

A third property is derived from the liquid limit and the plastic limit which is very useful in classifying fine-grained soils. The property is called the *plasticity index* and is designated PI. The PI is computed by:

$$PI = w_L - w_P \tag{4-4}$$

Physically, the plasticity index is the range of water contents over which a soil remains plastic. A soil having a low plasticity index will pass from its solid state through the plastic state and into the liquid state with the addition of only a small amount of water. A high-plasticity soil will require very large amounts of water to make this same transition all the way from solid state to liquid state. The size of the plasticity index is a key indicator of the plasticity (or compressibility) of fine-grained soils.

The determination whether a soil is a silt or a clay is based on its plasticity. The plasticity chart for fine-grained soils shown in Fig. 4-7 was developed in 1948 by the soil scientist Arthur Casagrande[10]. The chart is based on the molecular activity of the particles, a distinguishing property of silts and clays. Soils of larger size and lower molecular activity (silts) fall below the A-line; soils of smaller size and higher molecular activity (clays) fall above the A-line.

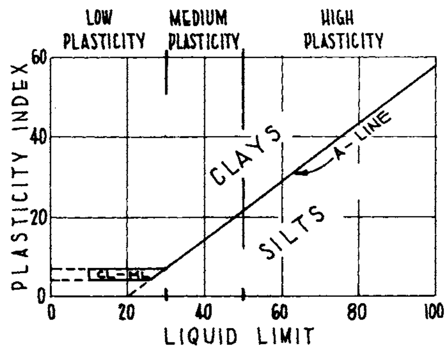


Figure 4-7 Casagrande Plasticity Chart

To determine whether a soil is a silt or a clay, enter the chart with the plasticity index and the liquid limit of the fine-grained soil. If the resulting point falls above the A-line, the soil is a clay. If the point falls below the A-line, the soil is a silt.

A further description of fine-grained soils can be made, based on the relative size of the liquid limit. Fine-grained soils, either silts or clays, that have a liquid limit above 50 are said to be of high plasticity (or of high compressibility). If the liquid limit is below 50, the soil is said to be of low plasticity (or low compressibility). The further distinction of a medium plasticity, as shown in Fig. 4-7, is not widely used.

The PI of a soil can also be used as a quick indicator of the susceptibility of a fine-grained soil to swelling (or expansion) when it is exposed to water. As a general indicator, a fine-grained soil having a PI greater than 25 should be regarded as a potentially "expansive" soil. If the PI is 50 or more, the soil should definitely be regarded as an "expansive" soil until proven otherwise.

Consistency and Textural Classification of Soils

There is an older judgemental type of soil classification that has been in use for many years and is still widely used. Its primary use is in describing a soil verbally. This classification utilizes either the texture of sands or the consistency of clays as shown in Table 4-4. In the table, sands and clays in the same general strength group are entered on the same line.

Table 4-4 Consistency and Textural Classification

Sands: textural classification	Clays: consistency classification	Estimate of bearing capacity kips/ft^2
---	Very soft	Less than 0.5
Very loose	Soft	0.5 to 1.0
Loose	Medium	1.0 to 2.0
Medium	Stiff	2.0 to 4.0
Dense	Very stiff	4.0 to 8.0
Very Dense	Hard	Over 8.0

CLAYS

1. A very soft clay can easily be penetrated several inches by fist.
2. A soft clay can be penetrated several inches by thumb.
3. A medium clay can be penetrated several inches by thumb with moderate effort.
4. A stiff clay can be indented readily by thumb, but penetrated only with great effort.
5. A very stiff clay can be indented readily by thumbnail.
6. A hard clay can be indented with difficulty by thumbnail.

SANDS

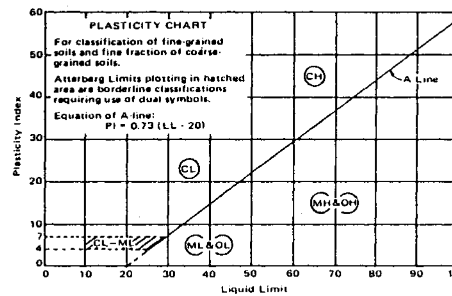
1. A very loose sand can be penetrated easily by a $^1/_2$ inch reinforcing bar pushed by hand.
2. A loose sand can be penetrated with difficulty by a $^1/_2$ inch reinforcing bar pushed by hand.
3. A medium sand can be penetrated readily by a $^1/_2$ inch reinforcing bar driven by a 5 lb hammer.
4. A dense sand can be penetrated about 1 ft. by a $^1/_2$ inch reinforcing bar driven by a 5 lb hammer.
5. A very dense sand can be penetrated about 3 inches by a $^1/_2$ inch reinforcing bar driven by a 5 lb hammer.

The textural and consistency classifications are still widely referenced in building codes and in older legal documents. Obviously, the classification is heavily dependent on the experience and bias of the person performing the test. Even so, it is a useful tool and is still widely used in practice.

Engineering Classification of Soils

Based on the grain size distribution and the Atterberg limits presented in the preceding sections, a standard classification system has been developed for the broad soil groups. The system has been adopted by ASTM under its designation D2487. Called the *Unified Soil Classification System*, this system is probably the most widely accepted classification system in current use. The Unified Soil Classification System is chart-based, as shown in the chart of Table 4-3.

Table 4-3 UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM)

Major Divisions			Group Symbols	Typical Names	Classification Criteria		
Coarse-Grained Soils More than 50% retained on No. 200 sieve*	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	Classification on basis of percentage of fines Less than 5% pass No. 200 sieve GW, GP, SW, SP More than 12% pass No. 200 sieve GM, GC, SM, SC 5% to 12% pass No. 200 sieve Borderline classification requiring use of dual symbols	$C_u = D_{60}/D_{10}$ Greater than 4 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines		Not meeting both criteria for GW	
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures		Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Atterberg limits plot above "A" line and plasticity index greater than 7	
	Sands More than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$ Greater than 6 $C_z = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines		Not meeting both criteria for SW	
		Sands with Fines	SM	Silty sands, sand-silt mixtures		Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Atterberg limits plot above "A" line and plasticity index greater than 7	
Fine-Grained Soils 50% or more passes No. 200 sieve*	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands			
			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 sands or silts, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity			
Highly Organic Soils			PT	Peat, muck, and other highly organic soils	Visual-Manual Identification, see ASTM Designation D 2488.		

*Based on the material passing the 3-in. (75-mm.) sieve.

Note that the most prominent subdivision in the "Major Divisions" column in the chart is that between the coarse-grained soils (gravels and sands) and the fine-grained soils (silts and clays). Note also that the classification of gravels and sands is based on the *coarse fraction* of the sample (the amount retained on the No. 200 sieve) rather than the total sample.

The soil classification itself consists of two letters, shown in the column headed "Group Symbols" in Fig. 4-3. The first letter designates the dominant type of soil particles:

G indicates gravel
S indicates sand
M indicates silt
C indicates clay

The second letter of the classification is descriptive. For clean gravels and sands (less than 5% fines), the second letter indicates whether the gravel or sand is well graded (GW or SW) or poorly graded (GP or SP). If the gravel or sand contains large amounts of fines (more than 12%), the second letter indicates whether the fines are silts (GM or SM) or clays (GC or SC). If the amount of fines is between 5% and 12%, dual symbols are used (e.g. GP-GM).

For fine-grained soils having a liquid limit above 50, the second letter of the group symbol indicates high plasticity (MH or CH). For fine-grained soils having a liquid limit below 50 and a PI above 7 or below 4, the second letter indicates low plasticity (ML or CL). For soils of low plasticity having a plasticity index between 4 and 7, a dual symbol is used (e.g. CL-ML). Note that any soil having a PI less than 4 is classified as a silt.

A few examples will illustrate the use of the classification system. Whenever a classification is being made, close attention must be paid both to the sieve sizes and whether the given information is for percent passing or percent retained.

Example 4-1 Soil Classifications in the Unified Soil Classification System

Given the following criteria, classify the soils

	Percent passing indicated sieve			
Sieve size	Sample 1	Sample 2	Sample 3	Sample 4
3 in.	100	100	90	100
1 in.	100	100	60	97
No. 4	95	100	44	62
No. 10	88	88	30	41
No. 40	81	76	16	33
No. 100	74	65	10	20
No. 200	65	54	6	13
Liquid limit	53	16	nonplastic	21
Plastic limit	36	10	12	

SOLUTION:

Sample 1 has 65% of its weight passing the No. 200 sieve; it is therefore classified as a fine-grained soil, either a silt or a clay. Its liquid limit is 53 and its PI is 17. From the A-line chart it is found to be a silt of high compressibility, designated MH.

[Note, however, that this "silt" contains 5% gravel (the material retained on the No. 4 sieve) and 30% sand (the material between No. 4 and No. 200 sieves). Nonetheless, this soil still falls in the soil group characterized by the large amount of silt in the mixture and is classified overall as a silt.]

Sample 2 has 54% of its weight passing the No. 200 sieve; it is therefore classified as a fine-grained soil, either a silt or a clay. Liquid limit is 16, PI is 6. From the A-line chart, the soil is found to fall in the special group of fine-grained soils given a dual symbol, CL-ML.

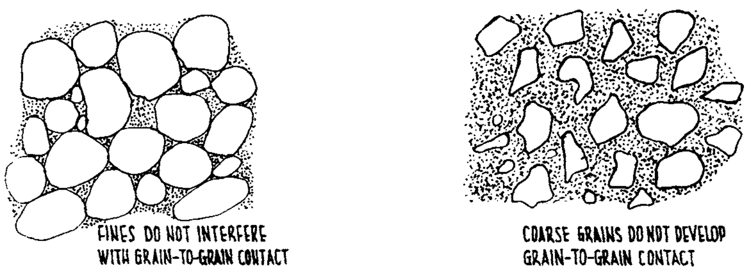
Sample 3 has only 6% fines; it is coarse-grained. The coarse fraction is taken to be 94 lbs of a 100 lb sample. The gravel in a 100 lb sample would be that part retained on a No. 4 sieve, or 100 - 44 = 56 lbs. The gravel portion of the coarse fraction is therefore 56/94 or 60% gravel. The coarse fraction is thus more than 50% gravel so the soil is a gravel. The 60% diameter D_{60} is seen to be 25 mm, the 30% diameter D_{30} is 2 mm and the 10% diameter is 0.15 mm. The coefficient of uniformity is then $C_u = D_{60}/D_{10} = 167$. The coefficient of curvature $C_z = (D_{30})^2/D_{60}D_{10} = 1.1$. Since C_u is greater than 4 and C_z is between 1 and 3, the gravel is found to be well graded, with fines between 5%

and 12% of its weight. Since the fines are nonplastic, the fines are automatically classified as silt. The soil is therefore classified GW-GM.

Sample 4 has only 13% fines; it is coarse-grained. Its coarse fraction is taken as 87 lbs of a 100 lb sample, with 38 lbs of the sample being gravel. The gravel portion of the coarse fraction is therefore 38/87 = 44%, or less than 50%; the soil is a sand. The sample has more than 12% fines with the fines having a liquid limit of 21 and a PI of 9. From the A-line chart, the fines are found to be clay. The soil falls in the broad grouping of clayey sands, SC.

One may wonder how a soil containing very high percentages of one soil group can be classified so absolutely in another soil group. The basis for the classification is in the way the soil responds to load. If the soil responds to load in the way that a sand responds to load, it is classified as a sand. If it responds to load in the way a clay responds to load, it is classified as a clay. The first letter of the classification therefore indicates immediately how the soil is going to respond to load, whether as a gravel, a sand, a silt or a clay.

The classification thus reveals immediately how a particular soil will carry a foundation load. A gravel or a sand, for example, will carry load by grain-to-grain bearing as indicated in Fig. 4-8a. As long as the amount of fines in the void spaces is so small it does not interfere with this grain-to-grain contact, the soil carries loads as a gravel or a sand and is so classified.



a) Coarse-grained soils b) Fine-grained soils

Figure 4-8 Load Transmission in Soils

Eventually, however, the amount of fines can increase to the point that the gravel or sand particles are no longer in grain-to-grain contact. Rather, they become isolated particles in a matrix of fines as indicated in Fig. 4-8b. At that point, the soil responds to load as a fine-grained soil and is then classified as a silt or clay.

As one may suppose, the point at which the soil mixture stops carrying load as a gravel or a sand and starts carrying load as a silt or a clay is not a precisely defined point. The ASTM classification shows the changeover occuring at 50% by weight, a nice round number. But while there may be a degree of vagueness in defining the actual point of changeover, the ASTM classification has, over the years, served as a valid indicator. It may therefore be used with certain degree of confidence.

Index Properties of Soils

There are other properties of a soil, called *index properties*, that are useful indicators. One such index property, the water content w, has already been introduced in Equation (4-3). There are others, one of the more important being the void ratio e, defined as

$$e = \frac{V_V}{V_S} \tag{4-5}$$

where V_V is the volume of voids in a sample and V_S is the volume of solids.

Another useful indicator is the porosity n, defined as

$$n = \frac{V_V}{V} \tag{4-6}$$

where V_V is the volume of voids in a sample and V is the total volume.

Another common indicator is the degree of saturation $S\%$,

$$S\% = \frac{V_W}{V_V} \times 100 \tag{4-7}$$

where V_W is volume of water and V_V is volume of voids.

The density of a soil is the same as it is for other solids,

$$\gamma = \frac{W}{V} \tag{4-8}$$

where W is the weight of a sample and V is its volume.

At times it is necessary to know the dry density of a soil, which is its density with all the water driven off. Dry density is denoted γ_d and is computed by

$$\gamma_d = \frac{W_S}{V} \tag{4-9}$$

where W_S is the weight of solids in a sample and V is the total original volume before the water was driven off.

The specific gravity of solids, G_s, is the specific gravity of the soil minerals, excluding air, gas and water. It ranges from about 2.3 to 3.0 but usually falls between 2.5 to 2.8 for most soils. The specific gravity of the solids in a silica sand, for example, is typically about 2.63 to 2.68. The unit weight of the solids,

γ_S, (having no voids at all) is computed as the unit weight of water, γ_W, times the specific gravity G_S, where

$$G_S = \frac{\gamma_S}{\gamma_W} \tag{4-10}$$

A somewhat imaginary quantity called *saturated unit weight* sometimes appears in soil calculations. The saturated unit weight, denoted γ_{SAT}, is defined as the unit weight of the soil when all voids are filled with water at *neutral pressure*. The term neutral pressure means that the porewater does not have any positive static pressure nor does it have any negative capillary tension. The saturated unit weight is computed as:

$$\gamma_{SAT} = \frac{V_V \gamma_W + W_S}{V} \tag{4-11}$$

Computation of the foregoing index properties can be simplified somewhat by using the following approach. A typical soil sample conformed to this approach is shown in Fig. 4-9. All the soil is shown lumped together as a separate mass, the water is shown as another separate mass and the air is shown as a separate volume.

When making computations using the diagram of Fig. 4-9, volumes are shown on the left of the diagram and weights are shown on the right. Air is considered to be weightless compared to the water and soil masses. To go from one side of the diagram to the other, it is necessary either to multiply or divide by the unit weight of water, 62.4 pcf, or by the unit weight of soil solids, 62.4G_S.

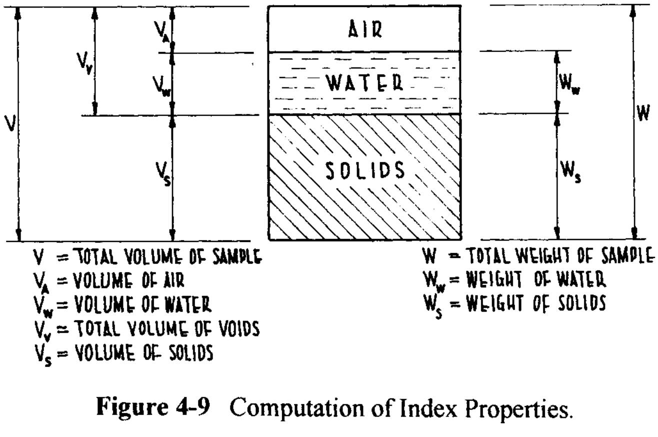


Figure 4-9 Computation of Index Properties.

Some examples will illustrate the computation of index properties using the format shown in Fig. 4-9.

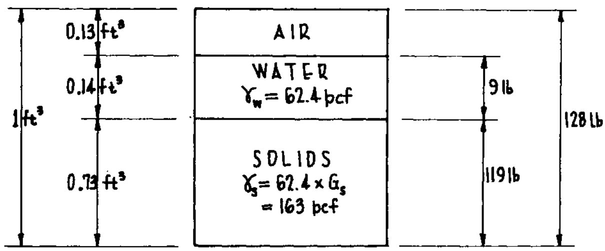
Example 4-2 Computation of index properties

Given : In-place density = 128 pcf, Dry density γ_d = 119 pcf,
Specific gravity of solids G_s = 2.61

To Find: void ratio e and degree of saturation $S\%$

Solution:

The sample is shown in the following diagram. Since density is given in lbs/cubic foot, a volume of 1 ft^3 is arbitrarily chosen as the sample size. The density and dry density then become weights, shown on the right side of the diagram.



The weight of the water, 9 lbs, is found as the total weight of the sample, 128 lbs, less the dry weight of the solids, 119 lbs. The volume of the 119 lbs of solids is found by dividing by the unit weight of the solids, 163 pcf; the resulting volume is entered on the left. Similarly, the volume of the 9 lbs of water is found by dividing by the unit weight of water, 62.4 pcf, with the result again being entered on the left. The remaining volume of the total of 1 ft^3 is the volume of the air, 1.00 - 0.73 - 0.14 = 0.13 ft^3.

The index properties can now be computed. The void ratio e is found by its definition,

$$e = \frac{V_V}{V_S} = \frac{0.13+0.14}{0.73} = 0.37$$

The degree of saturation is also found by its definition,

$$S\% = \frac{V_W}{V_V} \times 100 = \frac{0.14}{0.27} x100 = 52\%$$

All other index properties can be calculated similarly.

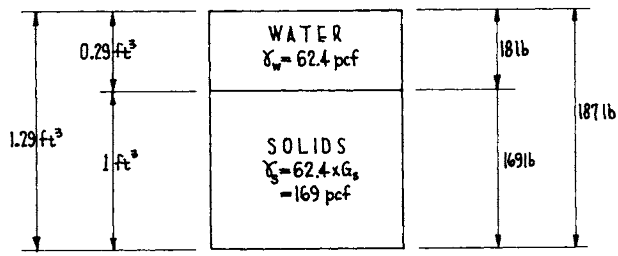
Another example will illustrate the procedure when the sample is saturated, that is, when all voids are filled with water at neutral pressure.

Example 4-3: Computation of Index Properties

Given : Saturated soil sample, Void ratio $e = 0.29$,
Specific gravity of solids $G_S = 2.71$

To Find: Porosity n and dry density γ_d

Solution:



The sample is shown in the foregoing sketch with the volume of air being zero. Since the void ratio is 0.29, the volume of solids is chosen as 1 ft^3; the volume of voids is then 0.29, entered on the left as shown. The total volume is simply the sum of the two. The weight of solids is found by multiplying the volume of solids, 1 ft^3, by the unit weight of solids, 169 pcf; the result is entered on the right. The weight of water is similarly found by multiplying the volume of water, 0.29 ft^3, by the unit weight of water, 62.4 pcf, with the result entered on the right. The total weight of the sample is simply the sum of these two weights.

The porosity n is found by its definition,

$$n = \frac{V_V}{V} = \frac{0.29}{1.29} = 0.22$$

The dry density γ_d is also found by its definition,

$$\gamma_d = \frac{W_S}{V} = \frac{169}{1.29} = 131 \text{ pcf}$$

The following example illustrates one of the minor mathematical complications that can occur in calculations for index properties.

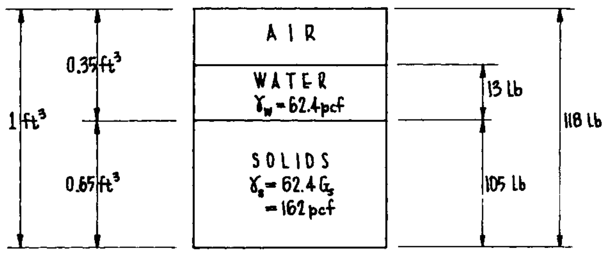
Example 4-4 Computation of Index Properties

Given : Water content $w = 12$, specific gravity of solids $G_S = 2.59$
Unit weight $\gamma = 118$ pcf

To Find: Void ratio e

Solution:

A weight of 118 lbs is entered on the following sketch for a sample having a volume of 1 ft^3.



A direct calculation for the volumes V_S and V_W in this case is not possible, since the weights W_S and W_W are not known. There are two equations, however, that contain the two unknowns W_S and W_W:

$$w = \frac{W_W}{W_S} \times 100 = 12 \qquad \text{or,} \qquad W_W = 0.12 \times W_S$$

and,

$$W = W_W + W_S = 118 \text{ lbs}$$

These equations are solved simultaneously, yielding;

$$W_S = 105 \text{ lbs}, \quad W_W = 13 \text{ lbs}$$

The volumes are now found readily:

$$V_S = \frac{W_S}{\gamma_S} = \frac{105}{162} = 0.65 \text{ ft}^3$$

$$V_V = V - V_S = 1.00 - 0.65 = 0.35 \text{ ft}^3$$

The void ratio is then:

$$e = \frac{V_V}{V_S} = \frac{0.35}{0.65} = 0.54$$

In general, the computation of index properties is readily accomplished when the format of the foregoing examples is used. Without such a format, the computations can become quite confusing.

Review Questions

4.1 Name the four broad soil groups.

4.2 Organic soils such as peat and lignite are not included in standard engineering classifications. Why not?

4.3 What is fragmentation?

4.4 What are some of the stronger influences that are continually breaking rock into grains of soil?

4.5 What influences can produce chemical changes in rock rather than mechanical breakage?

4.6 Why is soil so likely to be stratified?

4.7 Describe the soils in the A, B, and C horizons.

4.8 What feature makes a soil well graded?

4.9 When is a soil said to be uniformly graded?

4.10 Why are clays and silts not described as being well graded or uniformly graded?

4.11 Give the sieve sizes that fall at natural breakpoints between cobbles and gravels, between gravels and sands, between coarse sands and fine sands, and between coarse-grained soils and fine-grained soils.

4.12 What is meant by the D_{30} size of a soil?

4.13 What are the uniformity coefficient and the coefficient of curvature? How are they derived?

4.14 What values of C_U and C_Z would indicate a well-graded gravel? A well-graded sand?

4.15 Describe the physical meaning of the liquid limit.

4.16 Describe the physical meaning of the plastic limit.

4.17 Describe verbally the plasticity index in terms of water content.

4.18 What is the "A-line" that is used in classifying soils?

4.19 What index property is used in defining liquid limit and plastic limit?

4.20 Soils having a liquid limit above a certain value are said to be highly compressible. What is that value?

4.21 What is the classification of a soil whose PI falls below the A-line and whose liquid limit is above 50?

4.22 What screen size separates gravels and sands from silts and clays?

4.23 What is meant by a textural classification?

4.24 To what kinds of soils does a consistency classification apply?

4.25 What is water content? Void ratio? Porosity?

4.26 What is meant by "neutral" porewater pressure?

OUTSIDE PROBLEMS

Problems 4.1 through 4.6. Plot the gradation curves on semilog scale for the given soils. Compute C_u and C_z for the coarse-grained soils.

	Percent passing the indicated sieve size					
Sieve size	Soil 1	Soil 2	Soil 3	Soil 4	Soil 5	Soil 6
3 in.	100	80	100	100	100	100
0.75 in.	100	58	100	100	88	100
No. 4	100	39	95	100	72	100
No. 10	100	28	90	100	54	100
No. 40	91	13	80	100	26	99
No. 60	87	9	75	92	19	97
No.200	73	0	62	6	10	91
w_L	67	——	31	19	31	81
w_P	25	——	28	19	31	35

Problems 4.7 through 4.12. Classify the soils of Problems 4.1 through 4.6 in the unified classification system

Problems 4.13 through 4.18. Classify the given soils in the unified system. Interpolate as necessary to obtain D_{60}, D_{30} and D_{10}.

Percent passing the indicated sieve size						
Sieve	Soil 13	Soil 14	Soil 15	Soil 16	Soil 17	Soil 18
3 in.	100	100	100	100	100	86
0.75 in.	99	87	100	100	84	59
No. 4	80	57	95	100	62	31
No. 10	53	39	85	100	49	11
No. 40	9	23	77	79	45	2
No. 60	5	20	74	75	42	0
No.200	2	14	72	59	31	0
w_L	——	54	84	73	36	——
w_P	——	39	62	15	33	——

4.19 A soil is described in a telephone conversation as a soft clay. What range of allowable bearing strength should be expected for a foundation on this soil?

4.20 A soil is described as a dense sand. What would be the description of a clay having the same general level of bearing strength?

4.21 A soil is classified as a medium sand. What range of values of allowable bearing strength should be expected for a foundation on this soil?

4.22 A stratum of sand is vibrated until it becomes more dense, going from a loose sand to a medium sand. What happens to its allowable bearing strength because of this vibration?

Problems 4.23 through 4.28. Compute the remaining index properties for each of the given soils.

	Soil 23	Soil 24	Soil 25	Soil 26	Soil 27	Soil 28
G_S	2.68	2.61	2.63		2.60	
w			18.0		21.0	
γ			119 pcf	124 pcf		118 pcf
γ_d						106 pcf
e	0.61	0.41		0.71		0.56
n						
$S\%$	Sat	46.0		Sat	Sat	

PART II

RESPONSE OF A SOIL MASS TO FOUNDATION LOADS

Chapter 5

STRENGTH AND PRESSURE DISPERSION IN SOILS*

Permeability, Effective Stress and Submergence

The voids in a soil mass form an interconnected labyrinth of passages, a maze of "pores" through which water may flow or migrate whenever a pressure gradient is imposed. As the grain sizes of the soil become smaller and smaller, these passages become more and more numerous, but also smaller and more restrictive. As the size of the passage becomes smaller, the flow of the porewater becomes slower as the frictional "drag" force between the water and the soil particles increases.

The *permeability* of a soil is its capacity to permit the flow of water through its pores when a pressure gradient is imposed. It is this flow of water through a soil mass that permits water wells to be recharged. It is this same flow that can create "quicksand" in some silts. It is the presence of this water under pressure that creates heave in basement floors and leaking in basement walls. For coarse-grained soils, the existence of this water can at times cause a significant loss of strength, a point that will be noted repeatedly in subsequent discussions.

The apparatus that is traditionally used to demonstrate permeability in soils is shown schematically in Fig. 5-1. In Fig. 5-1a, the soil sample is saturated but not submerged. The somewhat imaginary term *saturated* means that all pores are filled with water under *neutral pressure*. At neutral pressure, the porewater exerts neither negative capillary pressure on the soil particles nor positive buoyancy on them.

The overall average intergranular pressure $\overline{p}$, also called *effective pressure* or *effective stress*, is computed as:

$$\overline{p} = H\gamma_{sat} \tag{5-1}$$

where γ_{SAT} is the saturated unit weight and H is the height of the sample. It should be apparent that the actual grain-to-grain contact pressure will be considerably higher than this average effective pressure. The grain-to-grain

* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

contact pressures could in fact approach infinity as the points of contact become smaller and smaller.

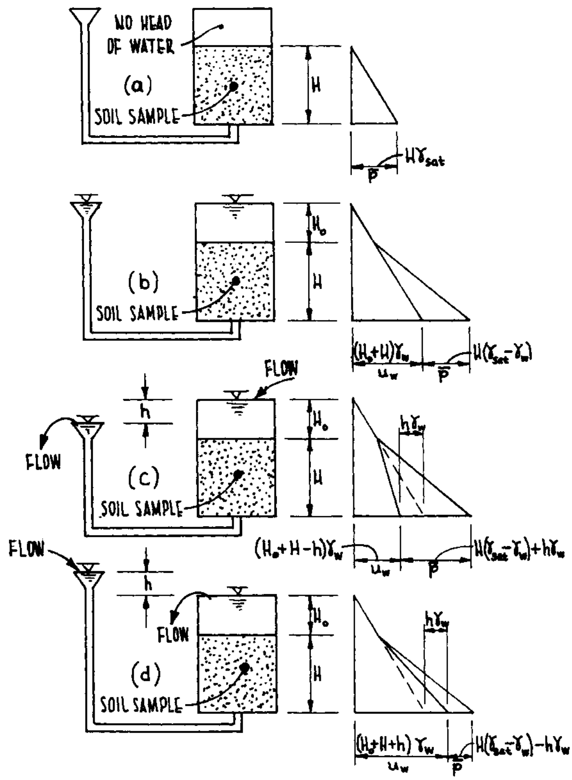


Figure 5-1 Effective pressure and Porewater pressure

Water is introduced as shown in Fig. 5-1b, but no flow of water is yet induced. The porewater is no longer at neutral pressure but has now developed a hydrostatic pressure as indicated. Under the hydrostatic pressure, the soil particles lose weight due to buoyancy. The loss in weight, in turn, causes a reduction in the average intergranular pressure $\bar{p}$ against the bottom of the container,

$$\bar{p} = H(\gamma_{sat} - \gamma_w) \tag{5-2}$$

where γ_w is the unit weight of water. Throughout the height of the sample, the porewater pressure u_W is simply the hydrostatic pressure in the water. Hence, at the bottom of the sample,

$$u_w = (H_0 + H)\gamma_w \tag{5-3}$$

where H_0 is the height of water above the top of the sample.

Typically, the unit weight of granular soils is about 115 to 135 pcf, or roughly twice the unit weight of water, 62.4 pcf. As indicated by Equation (5-2), the soil particles will therefore lose roughly half their original weight when submerged. The corresponding reduction in intergranular pressure is shown in the pressure gradients of Fig. 5-1b. This 50% loss in intergranular pressure will be shown later in this chapter to produce a corresponding 50% loss in strength in coarse-grained soils.

A downward flow of water is now induced as indicated in Fig. 5-1c. When the rate of flow reaches equilibrium, the hydrostatic head in the porewater at the bottom is readily computed at the left side, the outflow side in this case,

$$u_w = (H_0 + H - h)\gamma_w \tag{5-4}$$

where the height h represents the loss in pressure head between the inflow and outflow sides of the system. The height h is therefore the loss in head due to the frictional "drag" across the sample. Since the total pressure at the base of the container must remain unchanged, the effective pressure in the soil must increase by the amount of this drag. Hence,

$$\bar{p} = H(\gamma_{sat} - \gamma_w) + h\gamma_w \tag{5-5}$$

The downward flow of water therefore creates an increase in the average pressure between the soil grains due to the frictional drag on the grains.

An upward flow of water is now induced as indicated in Fig. 5-1d. The hydrostatic head at the bottom of the sample is again computed at the left side, the inflow side in this case,

$$u_w = (H_0 + H - h)\gamma_w \tag{5-6}$$

where the height h again represents the loss in head due to frictional drag across the sample. Since the total pressure at the base of the container must remain unchanged, the effective pressure in the soil must decrease by the amount of this drag. Hence,

$$\bar{p} = H(\gamma_{sat} - \gamma_w) - h\gamma_w \tag{5-7}$$

The upward flow of water therefore creates a decrease in the average pressure between the soil grains due to the frictional drag on the grains. The effective pressure has now been reduced by two effects, first by the buoyancy of the water and second by the uplift of the frictional drag.

For particular combinations of grain size, head loss, velocity of flow and unit weight, the intergranular pressure shown in Fig. 5-1d and expressed in Equation (5-7) can go to zero. In such circumstances, the soil particles become essentially weightless and the soil goes into a slurry called *quicksand*. In truth, however, the soil sizes most susceptible to this condition are silts rather than sands. Having the bottom of a dewatered excavation turn "quick" due to excessive pumping rates is not a rare phenomenon. The same phenomenon can also happen when flooded basements are pumped at high rates.

It might be suspected as well that the lateral pressures against the sides of the container in Fig. 5-1 are also undergoing changes as the vertical grain-to-grain pressures change. These changes in lateral earth pressures do in fact occur, although they are beyond the scope of an elementary text such as this. Standard references in lateral earth pressures treat the subject in detail[14], to put it mildly.

Measurement of the Shear Strength of Clays

The strength of a soil is determined much like the strength of any other structural material. A sample is taken of the material and tested to failure; the stress at failure is then said to be its *ultimate strength*. For use in design, a factor of safety is applied to this failure stress to obtain a "safe" allowable service stress, or allowable bearing pressure. The final design is then based on this allowable bearing pressure.

In making tests on soil, an additional influence occurs which does not occur in other materials. The actual field stratum is subject to a confining pressure laterally due to the overburden above. A typical soil stratum showing such an overburden pressure p_1 is shown in Fig. 5-2, with confining lateral pressure indicated as p_3.

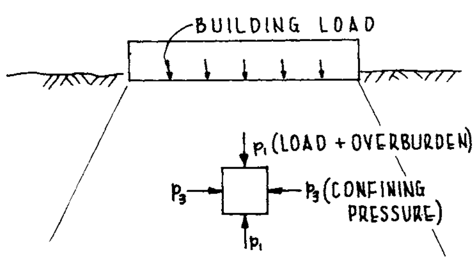


Figure 5-2 Lateral Confining Pressure in a Soil Stratum

Clay soils have enough cohesion that a cored cylinder of the soil may be extracted intact and tested in a triaxial compression test machine. Such a test is shown schematically in Fig. 5-3. The sample is encased in a membrane as shown, with the pressure p_3 reproduced in the surrounding watertight bell.

Field loading conditions are reproduced on the laboratory sample by pressurizing the bell until the confining pressure is equal to the field pressure p_3. A vertical pressure q_u is then added that reproduces the overburden pressure p_1 plus any increase in pressure Δp created by the foundation loads. The total pressure at the top of the sample is then $q_u + p_3$, where $q_u = p_1 + \Delta p$. As q_u is increased to failure, the lateral confining pressure p_3 may be varied as necessary to reproduce the field conditions.

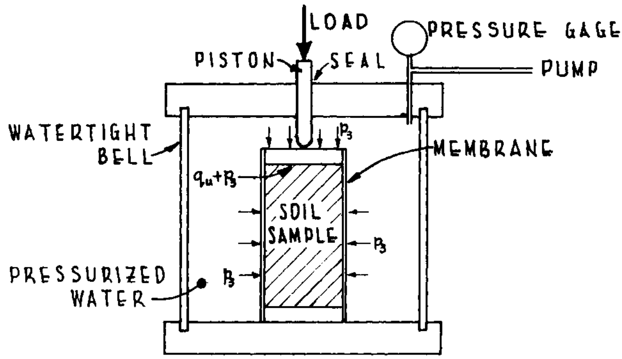


Figure 5-3 Schematic of a Triaxial Compression Test

In tests, it has been observed that a "pure" clay will fail along a 45° angle. (In the next chapter, this failure angle of 45° is shown analytically to be a general property of clays.) A prism of the sample has been removed and shown at larger scale in Fig. 5-4, with the resultant pressures in the two areas as shown. The area of the prism is designated simply A.

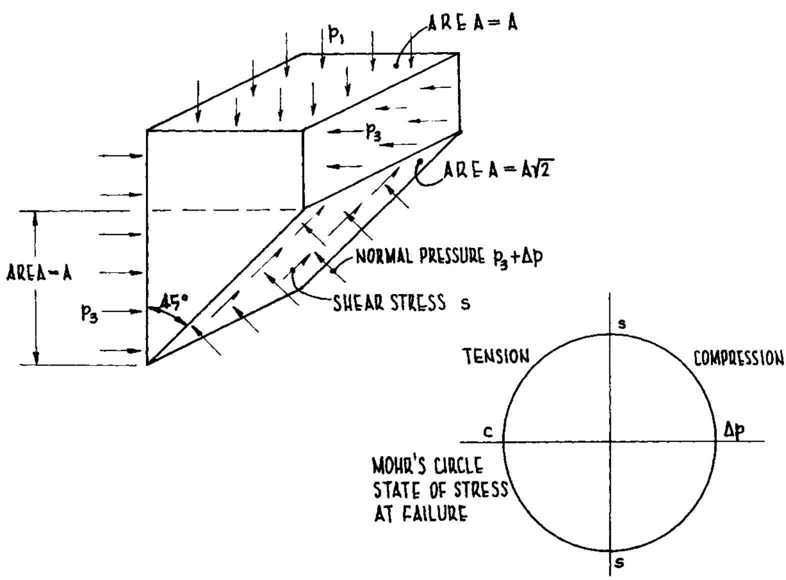


Figure 5 - 4 Stresses on the Sample

The vertical forces acting on the prism are summed, yielding

$$\sum F_V = 0 = p_1 A - \frac{(p_3 + \Delta p)A\sqrt{2}}{\sqrt{2}} - \frac{sA\sqrt{2}}{\sqrt{2}} \tag{5-8}$$

The result is simplified, yielding, with $p_1 = q_u + p_3$,

$$0 = q_u - \Delta p - s \tag{5-9}$$

The horizontal forces are summed:

$$\sum F_H = 0 = p_3 A - \frac{(p_3 + \Delta p) A \sqrt{2}}{\sqrt{2}} + \frac{sA\sqrt{2}}{\sqrt{2}} \tag{5-10}$$

When simplified, the result is:

$$\Delta p = s \tag{5-11}$$

Equation (5-11) is substituted into Equation (5-9) to find

$$s = \frac{1}{2} q_u \tag{5-12}$$

This surprisingly simple result states that the shear stress in the test sample at failure is 1/2 the applied vertical pressure q_u. It also reveals that the magnitude of the confining pressure p_3 has no effect on the shear strength when a pure clay is loaded "as is", that is, with no pretreatment of any kind. For this type of quick loading, there is no need to apply any confining pressure in making the test on a clay soil, since it does not affect the outcome.

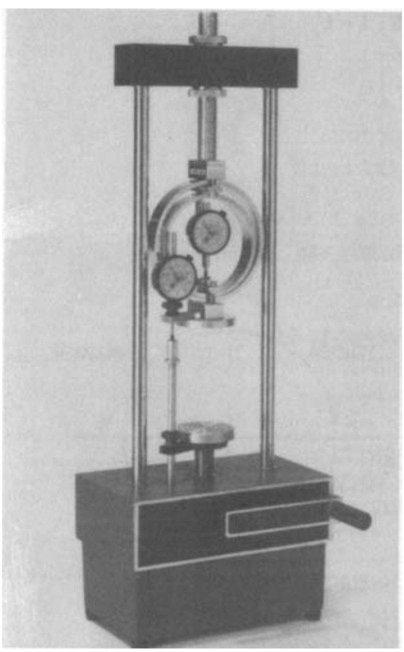


Figure 5-5 Unconfined Compression Test Machine (Photo courtesy ELE/Soiltest)

Since there is no need for confining pressure p_3, there is no corresponding need for membranes or pressure bells such as that in the triaxial test. The cylindrical sample of clay soil can simply be placed in a compression test machine and loaded to failure. (For those familiar with concrete testing, this test for the strength of a cylinder of a clay soil becomes directly parallel to the test of a standard concrete cylinder.) The test device then becomes quite simple, as shown in Fig. 5-5, requiring only a means to determine accurately the size of the clay cylinder and the load at failure.

The test shown in Fig. 5-5 is called the *unconfined compression test.* It is one of the more common lab tests in use today to determine the strength of a clay soil. The shear strength s is computed simply as $^1/_2$ the unconfined compressive strength q_u, or, $s = {}^1/_2\, q_u$.

The stress denoted c in the Mohr's circle of Fig. 5-4 is equal in magnitude to the increase in stress due to loads, Δp. It is called the *cohesion* of the soil and is usually denoted c. From the Mohr's circle, it may be observed that c is also the *tensile strength* of the clay soil at failure and is numerically equal in magnitude to the shear stress s at failure,

$$s = c \tag{5-13}$$

The unconfined compression test shown in Fig. 5-5 is widely used. The simple apparatus for running the test is available almost everywhere and is very easy to set up; the test is sometimes run on the tailgate of a jeep. Although test results are sensitive to irregularities in sampling, the test is cheap enough and simple enough that multiple tests may be run and erratic test results can simply be discarded.

The unconfined compression test just described is a special case of the triaxial compression test. In this case, the sample is not recompressed (consolidated) back to its in-place pressures before the test begins. The test is called an "unconsolidated quick test", since the entire failure load, which includes the overburden load, is applied so quickly there is no time for porewater to escape. The porewater is thus not at neutral pressure at any time during the test. As indicated in Fig. 5-6, the trapped porewater contributes to a marked reduction in strength.

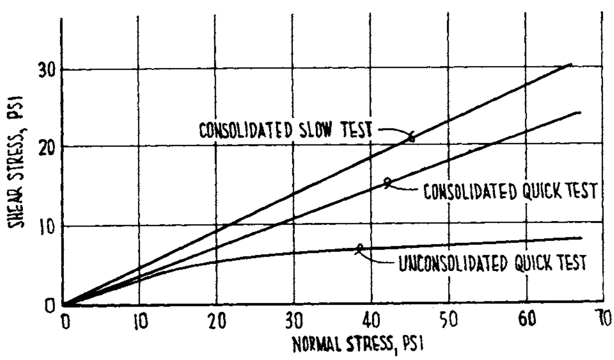


Figure 5 - 6 Comparison of Triaxial Test Results

There are two other variations of the triaxial test. In these tests, the overburden pressure is reapplied slowly such that the clay is recompressed (consolidated) back to its in-place service levels of pressure before the test begins. The porewater pressure is then at neutral pressure when the additional test load is applied.

- In the first of these variations of the triaxial test, the additional test load is applied quickly to the test sample, allowing no additional time for the porewater to escape (consolidated quick test). The trapped porewater is thus subject only to the added test pressure, simulating a suddenly applied load such as wind or earthquake on a structure already in service.
- In the second of these variations of the triaxial test, the additional test load is applied slowly, allowing time for porewater to escape as the pressure increases (consolidated slow test). The porewater is thus at neutral pressure (or nearly so) throughout the entire test. Such loading simulates a slow ever-increasing load on a soil, with no sudden buildup of porewater pressure.

Of the three tests, the "unconsolidated quick" test yields the lowest, most conservative values of strength. As indicated in Fig. 5-6, the slowly applied load in the "consolidated slow" test yields the highest values. The "consolidated quick" test falls somewhere between the other two. The difference in the results of the three tests is significant. As one might expect, the most conservative case is the one usually adopted (the unconsolidated quick test) since the field and service conditions are often unpredictable.

The unconfined compression test is at its most accurate in soils of high to medium plasticity. As the sand content in a soil mixture increases, however, the ability of the cylinder to stand alone decreases, reducing the applicability of the test.

In recent years, another test has been developed that is even more simple and which is gaining acceptance among engineers, especially among structural engineers. The test is called the vane shear test; one of the many devices currently used to perform a vane shear test is shown schematically in Fig. 5-7. The vane shear test is a field test, conducted in the actual soil stratum at its natural moisture content.

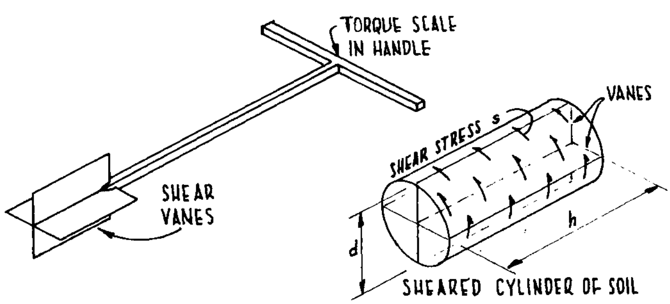


Figure 5-7 Vane Shear Test Device

In conducting the vane shear test, the vane is simply driven to the desired depth for a shallow test or placed in a predrilled hole for testing at depth. Torque is applied at the top of the rod until failure occurs; the torque T at failure is recorded. The shear strength at failure is then calculated using the applied torque at failure and the dimensions of the device. The solution of a simple calculation gives the shear strength of the soil:

$$T = \tfrac{1}{2} s\pi d^2 h + \text{ end effects} \tag{5-14}$$

where d and h are the diameter and height of the device

The vane shear test has the distinct advantage that since the test is run in place, no sampling errors can occur. Erratic results can sometimes occur, however, when a pebble or any other cause of inconsistency is encountered. In practice, several tests are run over a particular site and any erratic results are discarded.

It should be noted that the vane shear test measures the total shear strength s of a soil without regard to its classification as a clay or a sand. For clays with little or no sand content, the cohesion c will be close to this computed shear strength s. For mixtures of soils having a significant sand component, the computed shear strength s will still be valid, but there is no way to know how much of the shear strength s is due to friction in the sand or how much is due to cohesion in the clay.

The vane shear test has been found to be most accurate in plastic soils of soft to medium stiff consistency. It should be further noted that the test is valid only at the moisture content in the soil at the time the test was run.

Measurement of the Shear Strength of Sands

In a soil composed largely of sand, it is almost axiomatic that the strength of the soil is directly proportional to its relative density. In practice, improvements in the gradation of a sandy soil are often made by selectively blending it with another soil to increase its density. Such blending is directed toward filling more and more of the void spaces with correctly sized particles, thus improving the relative density of the mixture and thereby increasing its strength.

Conversely, any development that serves to "fluff" the sand will reduce its density and correspondingly reduce its strength. In the same vein, reducing the fines in a coarse-grained soil can sometimes reduce density and cause a loss in strength.

When a sand is being sampled for a strength test, it is important that the sand particles remain in their in-place orientation. Even small disturbances or vibrations can change the distribution of the particles, thereby changing the density of the soil and subsequently distorting the test. In sands, a representative sample of the soil must be undisturbed.

It is very difficult, however, to obtain an undisturbed sample of sand. The sand simply not cling together well enough to allow a sample to be extracted and handled. As more sophisticated methods are introduced to reduce disturbances, the cost of sampling and testing increases commensurately.

It should be apparent that the unconfined compression test that is used so widely for clays is unsuitable for sands. The triaxial test with its confining pressure will work, but its cost discourages its use. Further, there remains the probability that the sample will be disturbed anyway at some other point along the procedure.

For sands, a different type of test is usually made, the direct shear test. A typical apparatus for a direct shear test is shown in Fig. 5-8. The apparatus consists of two circular rings, aligned vertically, into which the sample of sand is placed.

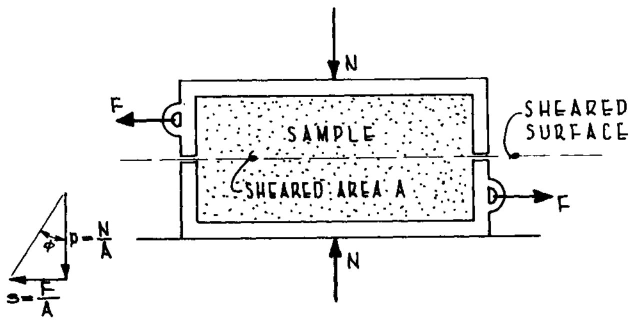


Figure 5-8 Direct Shear Test for Sands

Performing the direct shear test is quite simple. An overburden pressure p is applied to the sample. Then a shearing force F is applied at an ever increasing magnitude until the upper half of the sample is sheared sideways across the lower half. The shearing stress at failure is computed as the shearing force F divided by the area of sand undergoing the shear stress.

The shear stress at failure will of course increase as the overburden pressure p is increased. Unlike clays, the strength of sand is thus directly proportional to the amount of overburden, or overpressure. Curiously, as a footing load on sand increases, the increase in load produces an increase in the strength of the underlying sand; as load increases, strength increases. Such an increase in strength will continue until actual fracture of the sand grains begins.

The analogy between the direct shear apparatus and the friction devices used in high school physics is readily apparent. A comparison is shown in Fig. 5-9. The friction line between block and table (from high school physics) is seen to be directly analogous to the friction line between the upper and lower halves of the direct shear apparatus.

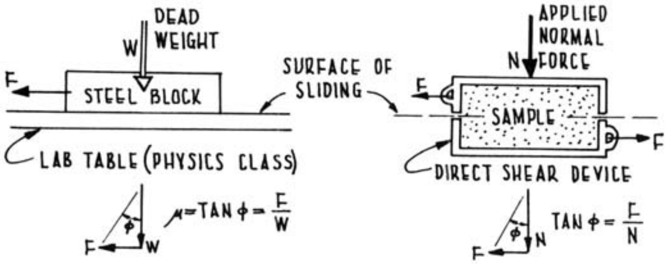


Figure 5-9 Friction analogy to the Direct Shear Test

The friction force is the coefficient of friction μ times the normal force N:

$$F = \mu N \tag{5-15}$$

The coefficient μ is thus the tangent of the angle ϕ., as shown in Fig. 5-9:

$$\tan\phi = \mu = \frac{F}{N} \tag{5-16}$$

In soil mechanics, Equation (5-16) is written in terms of stresses, or pressures, rather than forces. If the forces F and N in Equation (5-16) are divided by the sheared area, the resulting equation is then expressed in terms of stresses:

$$s = p\tan\phi \tag{5-17}$$

where s = shear strength at failure
p = average vertical intergranular pressure
ϕ = angle between s and p

The angle ϕ is a measure of the "roughness" of the sand particles and their tendency to interlock. The angle ϕ is called the *angle of internal friction.*

The angle of internal friction is a direct measure of the strength of the sand. A sand in its most dense state can have an angle of internal friction up to 40° or more. A sand in its loosest state can have an angle of internal friction down to about 26°. The end result of the direct shear test is the determination of this angle of internal friction.

It should be evident from Equation (5-17) that the magnitude of the shear stress s at failure is dependent on the magnitude of the intergranular pressure p. If p increases, s increases. One may conclude that the strength of a sand will increase in any circumstance where confining pressure increases, including the sand at the lower depths of a sand stratum, or the sand under a loaded footing.

The direct shear test shown in Fig. 5-9 has been widely used for many years. Even so, it suffers from the same disadvantage as other tests used to determine the angle of internal friction, the lack of an undisturbed sample. Even a slight jolt of the sample can sometimes change the results measurably.

In recent years, indirect test measurements by field testing devices have been developed that have all but replaced the direct shear test. In these indirect field tests, a device on the end of a rod is driven into the ground. The resistance offered by the soil to the penetration of the device is directly related to the strength of the soil. Two such devices are shown in Fig. 5-10.

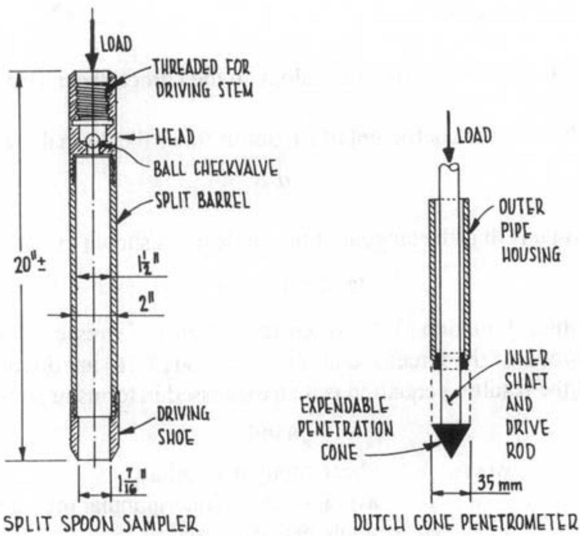


Figure 5-10 Standard Penetration Devices

In the United States, the oldest and best-known example of such indirect field tests is the *standard penetration test* (SPT). In the standard penetration test, the device is driven into the ground with a hammer. The SPT is classified as a dynamic test.

A more recent development is an analogous field test known as the *Dutch cone test*. In the Dutch cone test, the device is pressed slowly into the ground at a prescribed rate by a hydraulic ram. The test is classified as a static test.

The results of the SPT and the Dutch cone test can be directly correlated. There are, however, several variations of the Dutch cone test in use. At times, correlation may be difficult or impractical.

In the standard penetration test, a sampling device called a *split spoon sampler* is used. The split spoon sampler has long been used for the routine sampling of soils. It was eventually discovered, however, that the number of blows required to drive the sampler a foot into the ground to obtain a sample of soil was also a good indicator of the strength of the soil. The discovery was studied, tested and refined until it evolved into today's standard penetration test.

The standard penetration test is conducted by driving the split spoon sampler into the soil using a 140 lb hammer falling 30 inches. The number of blows required to drive the point one foot into the soil is the *SPT blow count N* for the test. The blow count *N* has been correlated by extensive testing to the angle of internal friction ϕ of the soil and to the Dutch cone resistance, as shown in Fig. 5-11. For consistency, the final value of the SPT blow count *N* obtained in the test is adjusted to an overburden pressure of 1500 lbs/ft^2.

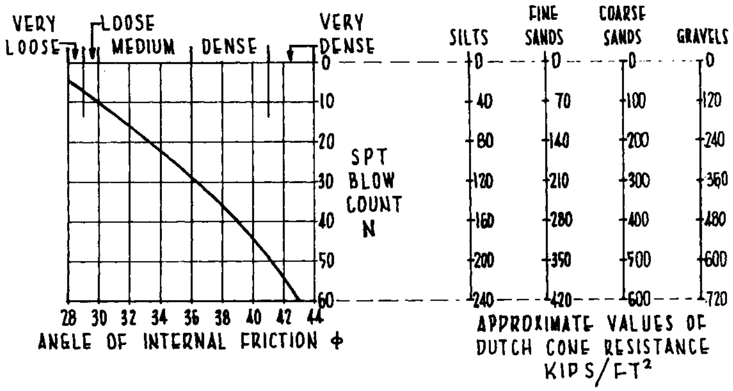


Figure 5-11 Graphic Correlation of Penetration Tests

The Dutch cone test is conducted by pressing the expendable cone shown in Fig. 5-10 into the soil at a rate of 2 cm /sec using a hydraulic ram. The results of the Dutch cone test are stated as the resisting pressure in Newtons/mm^2 or in lbs/in^2. Average values of Dutch cone resistance are included in Fig. 5-11 for coarse-grained soils.

Both the SPT and the Dutch cone test are most accurate in coarse-grained soils. They are often applied, however, to fine-grained soils. The accuracy of the results can be quite erratic in fine-grained soils and should be used with caution. Because the penetration tests are so universally used in fine-grained soils, however improperly, guideline values are presented for reference in Fig. 5-12. The chart gives an approximate correlation between blow count, textural classification, cohesion and unconfined compression. Such values can be useful when information is sparse and time is short.

The SPT values recorded in any series of field tests will necessarily be obtained at various depths of overburden. Since the strength of a sand is dependent on the vertical pressure, the variation in *p* in SPT tests is commonly *normalized* to a standard pressure. Several methods of normalizing the test results are available, usually to a pressure of 2000 lb/ft^2, but sometimes to a higher or lower pressure.

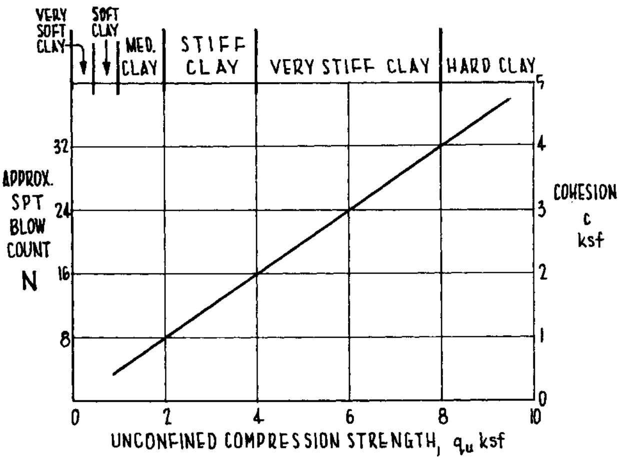


Figure 5-12 Penetration Test Values in Fine Soils

For use in shallow foundations, a normalization pressure somewhat lower than 2000 lbs/ft^2 is justified. Remember that the vertical pressure on the soil will include both the load on the footing as well as the load of the overburden above the depth of founding D_f. For shallow foundations, the depth of overburden D_f can vary from a minimum of 16 inches up to 12 feet or so. To this is added the increase in footing pressure Δp due to loads, which could be as low as 1200 to 1500 lbs/ft^2. At a minimum, the total in-service soil pressure might therefore be as low as 1500 lbs/ft^2. This value of 1500 lbs/ft^2 is selected here as a conservative but reasonable minimum pressure on which to normalize the soil strength.

At a normalization pressure of 1500 lbs/ft^2, the corresponding depth of a field test in a sandy soil weighing 125 lbs/ft^3 is about 12 ft, consistent with the limitations on D_f adopted earlier. The strength of the sandy soil, based on the field tests, will generally be normalized to the specified pressure before being forwarded to the structural designers. Calculations using the normalized strength of the sandy soil will then be "safe" down to a minimum total footing pressure (overburden plus structure) of about 1500 lbs/ft^2.

The following equations[5] provide the normalized pressure N from the field test, where the field test is conducted at a vertical pressure of p_l to obtain a field blow count of N_{pl}. Usually, $p_l = \gamma h$, where γ is the unit weight of the soil and h is the depth at which the test is made.

$$500\ \text{psf} < p_1 < 1500\ \text{psf}, \qquad N = N_{p_1} \frac{2000}{500 + p_1}$$

$$1500\ \text{psf} < p_1 < 6000\ \text{psf}, \qquad N = N_{p_1} \frac{8000}{6500 + p_1} \qquad (5\text{-}18, 19)$$

A graph showing the parameters of Equations (5-18) and (5-19) is shown in Fig. 5-13. Note that the pressure p_l is given in lbs/ft^2. The approximate depth of the test is given for reference only; it is based on a unit weight of a sandy soil of about 125 lbs/ft^3.

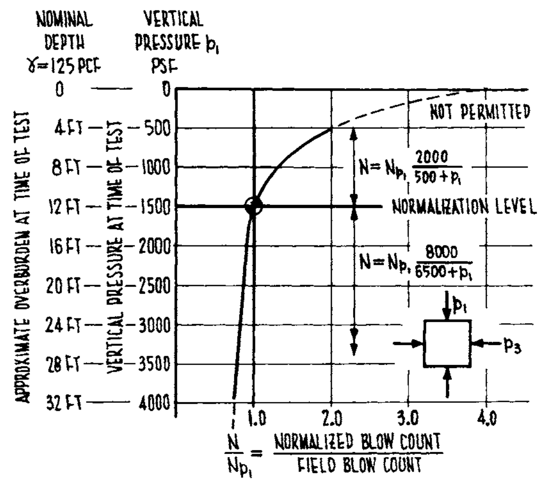


Figure 5-13 Normalization of SPT Blow Counts

As in the field vane shear test for clay soils, the field penetration tests for sandy soils yield only the overall strength of the soil. There is no way to distinguish how much of this strength is due to the sand component, if any, nor how much is due to the clay component, if any.

Some examples will illustrate the use of field test results and normalization practices to obtain values that can be used for design.

Example 5-1 Comparison of test results in a sandy soil

Given : Dutch cone resistance q_c in a fine sand is reported from the field test results to be 94 kips/ft^2.

To find: The equivalent SPT blow count and the normalized angle of internal friction.

Solution:

From Fig. 5-11, the equivalent SPT blow count is found to be about 13. Since the depth at which the test was made is unknown, the normalized value of the blow count N cannot be determined from the given information. The normalized angle of internal friction cannot therefore be determined.

Example 5-2 Comparison of test results in a sandy soil

Given : Normalized SPT blow count of a coarse sand is found to be 15.

To find: The comparable Dutch cone resistance and the angle of internal friction ϕ of the sand.

Solution:

From Fig. 5-11, the Dutch cone resistance comparable to a SPT blow count of 15 is found to be 150 for a coarse sand.

For a SPT blow count of 15, the angle of internal friction of the coarse sand is found to be roughly 32° when the total vertical pressure is about 1500 psf.

Example 5-3 Normalized SPT blow count in a sandy soil

Given : A field SPT value is reported to be 6 for a test taken at a depth of 5 feet in a fine sand having a unit weight of 118 pcf

To find: The normalized value of the angle of internal friction ϕ to be used in design.

Solution:

The normalization ratio for blow count N may be read from the graph of Fig. 5-12 or it may be computed from the formula given by Equation (5-18). The computed value is given by:

$$N = N_{p_1} \frac{2000}{500 + p_1} = 6 \frac{2000}{500 + 118 \times 5} = 11$$

From Fig. 5-11, the angle of internal friction corresponding to a normalized blow count of 11 is 30°.

Example 5-4 Normalized SPT blow count in a sandy soil

Given : A field SPT value is reported to be 16 for a test taken at a depth of 24 feet in a coarse sand having a unit weight of 121 pcf.

Empirical formulas for calculating the modulus of elasticity of a sand are given in terms of the Dutch cone resistance q_c:

Under square footings, $E = 2.5q_c$ (kips/ft^3)
Under strip footings, $E = 3.5q_c$ (kips/ft^3)

To find: 1) The normalized value of the angle of internal friction ϕ for design.

2) The modulus of elasticity of the soil under a square spread footing.

Solution:

The normalized value of the SPT blow count N is computed from the formula given by Equation (5-19):

$$N = N_{p_1} \frac{8000}{6500 + p_1} = 16 \frac{8000}{6500 + 121 \times 24} = 14$$

1) For a normalized blow count of 14, the angle of internal friction ϕ is found from Fig. 5-11 to be roughly 32°

The Dutch cone resistance of a sand corresponding to a SPT blow count of 14 for that sand is found from Fig. 5-11 to be 140 ksf.

2) The modulus of Elasticity of the soil is computed to be

$$2.5q_c = 2.5(140) = 350 \text{ kips/ft}^3.$$

Example 5-5 Comparison of normalized SPT blow counts

Given : A field SPT test is made at a depth of 11 feet in a fine sand weighing 117 pcf; the blow count was 12.

At a later time, the entire area will be cut 4 feet and a building will be built with foundations placed 3 ft. below the new grade.

To find: 1) The normalized angle of internal friction in the sand at the original depth of 11 feet before the 4 feet of overburden is removed

2) The value of the normalized angle of internal friction after the 4 feet of overburden is removed

Solution:

Before the cut is made, the normalized blow count is given by Equation (5-18) with an overburden pressure p_l of $117 \cdot 11 = 1287$ psf:

$$N = N_{p_1} \frac{2000}{500 + p_1} = 12 \frac{2000}{500 + 1287} = 13.4$$

1) For a normalized blow count of 13.4 before the cut is made, the angle of internal friction under a normalized overburden pressure of at least 1500 psf is found from Fig. 5-11 to be about 31°

After the 4 feet of cut is removed, nothing would change. If a second field SPT test were to be performed at a depth of 11 feet, it would still indicate a blow count of 12 at 11 feet of depth in this sand. For a total vertical pressure not less than 1500 psf due to overburden plus footing load, the normalized angle of internal friction for this sand will always be 31°.

2) The normalized angle of internal friction after the cut is made will remain 31°, though it would improve with depth.

The Coulomb Equation for the Strength of Soils

As a point of review, the shear strength of a clay has been shown to be the numerically equal to the cohesive strength of the clay,

$$s = c \tag{5-20}$$

Similarly, the shear strength of a sand has been shown to be its frictional resistance,

$$s = p\ tan\phi \tag{5-21}$$

For pure sands or pure clays, it is thus quite simple to find the shear strength of either soil from the foregoing relations.

Most soils, however, are neither pure sands nor pure clays. As indicated in the classification chart of Table 4-3, most soils are mixtures of sands and clays, being distinguished only by the amount of material being retained or passed by the No. 200 sieve. The shear strength of such a mixture should obviously be some combination of the two.

The shear strength of a soil mixture can be taken to be the direct sum of its shear strength due to cohesion plus its shear strength due to friction:

$$s = c = p \tan\phi \tag{5-22}$$

While there may be some occasional inconsistencies in directly summing the two shear strengths, the result is generally adequate for the design of shallow foundations. Equation (5-20) is the Coulomb equation; it is the equation that defines the shear strength of general mixtures of clays and sands. In any general solution for the failure load or failure mechanism in a soil mass, the Coulomb equation is the governing equation.

Dispersion of Load into a Soil Mass

The dispersion of a foundation load into a soil mass is treated in the mathematical theory of elasticity as a concentrated load at the surface of a semi-infinite elastic solid. The mathematical solution to this problem was given by Boussinesq in 1885[5]. In the Boussinesq solution, the solid is presumed to be homogeneous, isotropic and elastic, with these properties extending indefinitely in all directions.

In the Boussinesq solution, a concentrated load P is assumed to act at the surface of the solid as shown in Fig. 5-14. The increase in vertical stress, $\Delta\sigma_z$, at a depth z below the surface and at a lateral distance r from the load P is given by the theory of elasticity. The equation is given below along with a defining sketch of its terms.

$$\Delta\sigma_z = \frac{3P}{2\pi}\left[\frac{z^3}{\left(r^2+z^2\right)^{\frac{5}{2}}}\right]$$

(5-23)

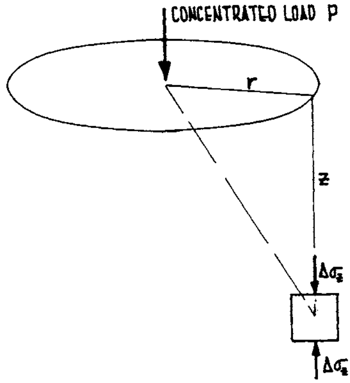
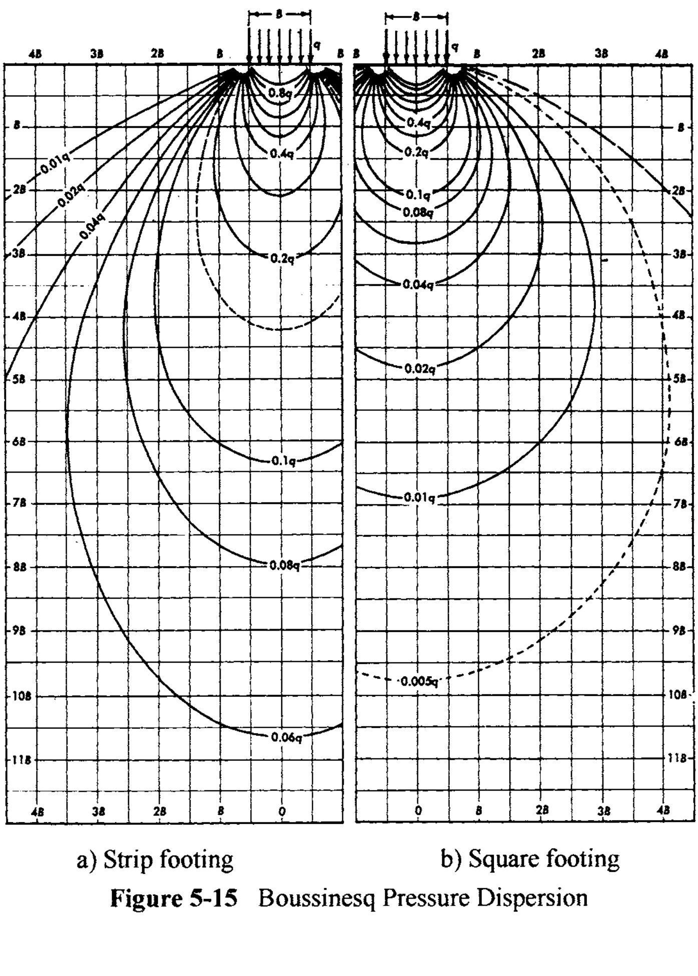


Figure 5-14 Stresses in a Semi-infinite Elastic Solid



a) Strip footing b) Square footing

Figure 5-15 Boussinesq Pressure Dispersion

The solution to Equation (5-23) is shown graphically in Fig. 5-15 for a footing of width B. At one side is the solution for a strip footing and at the other side is the solution for a square spread footing. The lines represent points of equal vertical pressure, given as a fraction of the contact pressure q. Dimensions are given as multiples of the footing width B.

It is common practice in soil mechanics to represent the multiplicity of three-dimensional Boussinesq contour bulbs of Fig. 5-15 with a single contour bulb as indicated in Fig. 5-16. In such an abbreviated representation, the outermost contour bulb is usually taken to be the $0.10p$ bulb for spread footings and $0.15p$ for strip footings. These values are within the usual accuracy for soils.

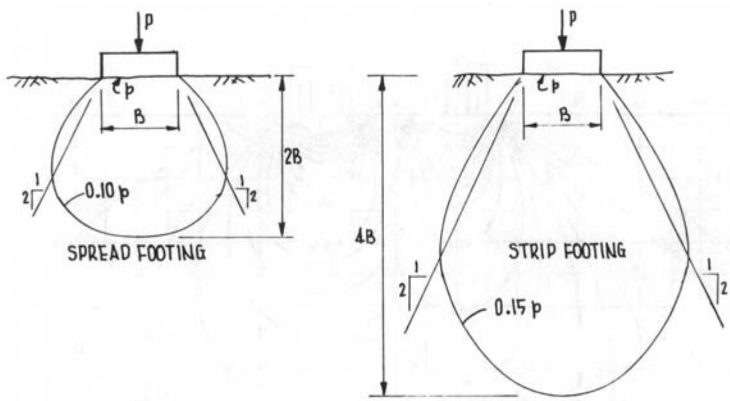


Figure 5-16 Abbreviated Boussinesq Pressure Bulbs

It is observed in Fig. 5-16 that the dispersion of pressure into the soil mass occurs at a nominal slope of about 2 vertical to 1 horizontal. This approximation of the dispersion angle has been used successfully in foundation design for many years. Its accuracy has been found to be within the general range of accuracy obtainable in soils.

In the pressure bulb for spread footings of Fig. 5-16, the pressure contour of $0.10p$ extends downward a distance of about 2B below the bottom of the footing. Similarly for strip footings, the pressure contour of $0.15p$ extends downward a distance of about 4B below the bottom of the footing, or roughly twice as deep. The difference occurs since the pressure under a spread footing can be dispersed laterally in all four directions, while under a strip footing the pressure can be dispersed only in two directions. It thus takes roughly twice as much depth to dissipate the soil pressure under a strip footing as it does under a spread footing.

The actual pressure variations corresponding to the Boussinesq solution are shown in Fig. 5-17. As one might expect, the pressure is highest at the centerline of load, diminishing as the load is dispersed outward and downward.

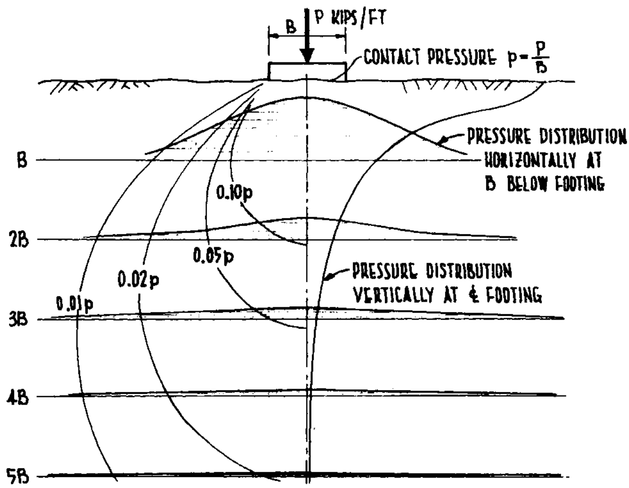


Figure 5-17 Boussinesq Pressure Variations

It should be noted immediately in the foregoing analysis that few soils could meet the requirements of the Boussinesq solution. Soil is not very homogeneous, nor is a soil likely to extend very far in any direction without a serious interruption in the little homogeneity that it has. Also, soil is not isotropic, that is, it does not have the same properties in all directions, to include tension as well as compression. However, if an entire soil mass experiences only an *increase in compression* due to bearing loads (no tension), it has been found that soil exhibits a reasonable degree of isotropy. Insofar as the elasticity of soil is concerned, it was observed earlier that at service levels of load, soil responds reasonably linearly under compressive load.

Even with these severe limitations, the Boussinesq solution is still applied to soils. The Boussinesq solution is not used as an absolute solution, however. Rather, it is used as a standard against which the pressure distribution in any soil can be compared (or approximated).

The three-dimensional pressure bulb that exists under an entire building may be visualized as shown in Fig. 5-18a, based on the overall average increase in pressure under the entire building footprint. Or, it may be visualized as shown in Fig. 5-18b, based on the much higher contact pressures under the individual footings. Either way, the dispersion of pressure at some depth well below the building is the same.

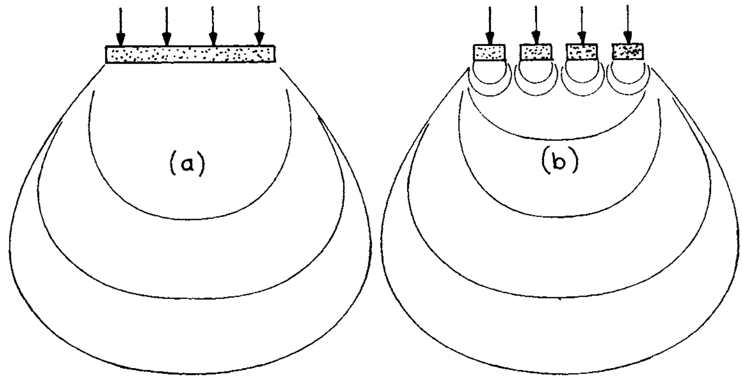


Figure 5-18 Overall Pressure Distribution

The actual Boussinesq pressure bulbs themselves are rarely if ever used in design practice. Even so, the concepts and parameters of the Boussinesq solution form the basis for most of the approximate methods that will be introduced later. Some examples will illustrate the use of the pressure bulbs in general as they apply both to overall building pressures and to individual footings.

Example 5-6 Pressure dispersion under a building

Given : Building 60 ft square that places an average pressure of 300 psf over the building footprint

To find: 1) depth below the foundation at which the average increase in soil pressure will be reduced to half the pressure at the top

2) depth below the foundation at which the average increase in soil pressure will be reduced to one-fourth of the pressure at the top

Solution:

From Fig 5-15, the pressure contour of $0.50p$ extends downward to about 0.75B and the contour of $0.25p$ extends downward to about 1.3B.

1) At a depth below the foundation of 0.75 x 60 or 45 feet, the increase in pressure will be about one-half that at the foundation level, or a 150 psf increase.

2) At a depth below the foundation of 1.3 x 60 or 78 feet, the increase in pressure will be about one-fourth that at the foundation level, or a 75 psf increase.

Example 5-7 Pressure dispersion under a footing

Given : Column footing 6 ft 3 in. square supporting a column load of 120,000 lb.

To Find: Depth below the footing at which the soil pressure will increase by 150 psf

Solution:

The contact pressure under the footing is found to be

$$120{,}000 \text{ lb.}/6.25^2 \text{ ft}^2 \text{ or, } 3070 \text{ psf.}$$

The increase in pressure is then 150/3070 or roughly 5% of the contact pressure.

From Fig. 5-15. the 5% contour is found to extend downward to 3.2B, or 3.2 x 6.25, or to about 20 ft.

Approximate Dispersion of Load into a Soil Mass

To simplify calculations, the Boussinesq pressure bulbs are usually approximated by the pyramid or wedge shapes shown in Fig. 5-19. These approximate shapes, however, are still called the pressure "bulbs" in all subsequent discussions. The existence of these pressure bulbs must be borne in mind at every stage in the design of foundations.

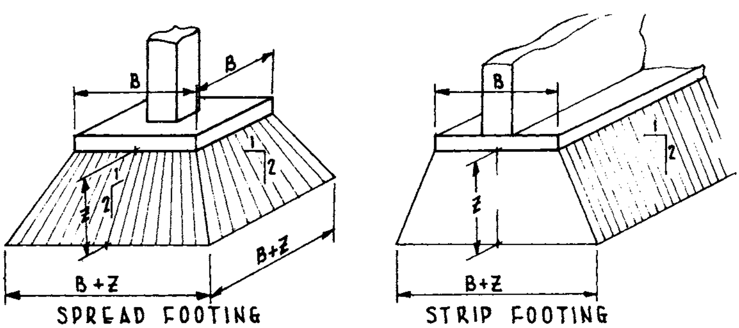


Figure 5-19 Approximate Pressure Bulbs

For the approximate pressure bulbs of Fig. 5-19, side slopes are taken at 2 vertical to 1 horizontal for all ordinary soils. In violation of the actual pressure variations shown earlier in Fig. 5-17, pressure at any horizontal plane is assumed to be uniform. The influence of pressures beyond the sloped sides and bottom surface is presumed to be negligible and can safely be ignored.

All calculations for settlements may be based on this average uniform pressure at each level below the founding line. Over the years, the use of this average uniform pressure in calculating average settlements has been found to produce acceptable results. It remains, however, only an approximation of the actual Boussinesq pressure variations shown in Fig. 5-17.

Where the pressure bulb extends into a second or a third or a fourth soil stratum having different characteristics from the first, any distortion of the approximate pressure bulb is considered to be small enough to be ignored. The dispersion angle remains 2 vertical to 1 horizontal for both soils with no appreciable distortion at their juncture.

The pressure under an isolated spread footing is dispersed in two directions and dissipates rapidly, as indicated earlier in Fig. 5-16. At a depth of 2B below the founding line, the pressure is already down to 10% of the contact pressure. In comparison, the pressure under a strip footing or grade beam disperses in only one direction and will penetrate to a much greater depth, up to 4B below the founding line.

Wherever the pressure bulbs overlap, a buildup of pressure occurs, with a commensurate buildup of settlements within the overlapping zones. Such overlaps can occur when footings are placed too close together, when large loads and their large footings are placed adjacent to small loads and their small footings, or where spread footings and strip footings are intermixed in the same foundation system. A sketch of such overlapping zones of influence is shown in Fig. 5-20.

In general, precise calculations involving such fine points as overlapping pressure bulbs are subject to question. The soil properties that must be used in such calculations are rarely consistent enough or accurate enough to permit such fine tuning. It is far preferable to eliminate the overlapping pressure zones shown in Fig. 5-20 than to try to deal with them. Rude but positive measures to circumvent problems with these and other such induced interactions are discussed in later chapters.

A valuable generality in foundation design may be deduced from the pressure bulbs of Fig. 5-20c. It is observed that the large footing with its large load causes compression over a much greater depth and volume of soil than the small footing does. It can be surmised (correctly) that *at equal contact pressures, a large footing will settle more than a small one.* Footing settlement S is therefore a function not only of stress p (pressure) but also of size B:

$$S = K_0 pB \tag{5-24}$$

where K_0 is the constant of proportionality.

This generality in the behaviour of foundations will be found in later discussions to be analytically correct, and as such, to be a useful indicator of high settlements.

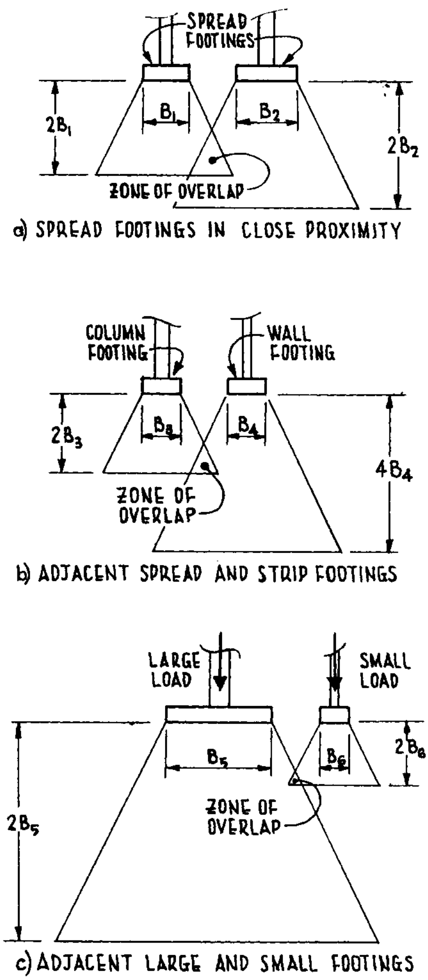


Figure 5-20 Overlapping pressure bulbs

Pressure Dispersion Through Underlying Strata

Stratification of a supporting soil is of course a common foundation condition. Tilted and uneven depths of stratification as shown in Fig. 5-21 is so common that they should be expected wherever stratification is present.

Where a pressure bulb crosses a stratum of compressible soil of variable thickness as shown in Fig. 5-21, the foundation will be subject to nonuniform settlement. The entire building will eventually tilt very slightly as the more highly loaded part of the stratum settles more than the more lightly loaded part.

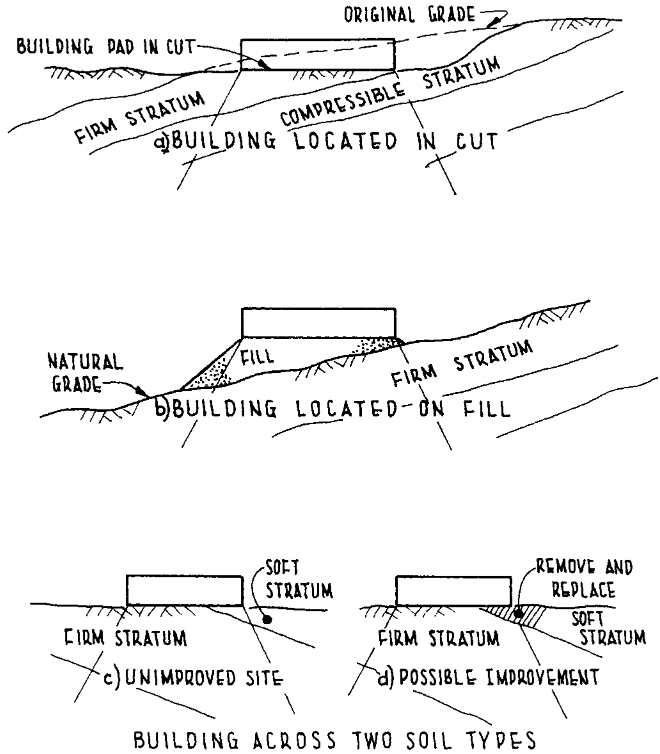


Figure 5-21 Typical Soil Stratification

The settlements of the buildings of Fig. 5-21a and b could still fall within the allowable limit of 1 inch and therefore be technically acceptable. Whether the design should include measures to equalize the settlements and reduce the slight tilt is left to the individual designer; the tilt is likely to be undetectable, it does no harm to the building and all settlements remain within specified limits.

The stratification of the soil of Fig. 5-21c, however, is a different matter. In this case, the settlement will not produce a linear tilt but a distinct increase in settlement at the right end of the building. Eventually, the building will undergo a "broken back" as the right side settles more than the left. Cases such as this are beyond the scope of an elementary text such as this, but a common remedy for such a problem is to remove the offending wedge of material and to replace it with a soil that is more compatible with the underlying stratum.

If the stratification is reasonably uniform as shown in Fig. 5-22, it can be expected that settlements at equal soil pressures will also be reasonably uniform. The soils report in such cases should specify the allowable soil pressure within each stratum. The foundation design would then include a check of the pressures at the bottom of each stratum to see that the allowable pressures are not exceeded.

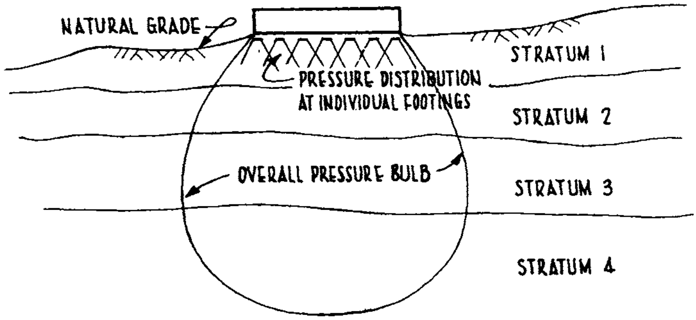


Figure 5-22 Uniformly Stratified Soil

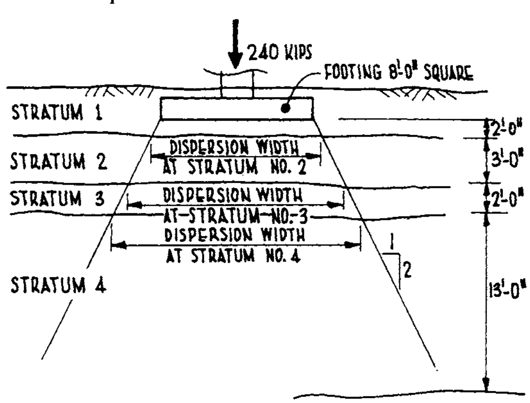
For the approximate pressure bulb used in the following example, side slopes are taken at 2 vertical to 1 horizontal as indicated earlier in Fig. 5-16. Pressure at any horizontal plane is assumed to be uniform. The load of 240 kips is constant throughout the depth of the pressure bulb; pressure decreases, however, as the dispersion area increases.

Example 5-8 Calculation of footing pressures with depth

Given : Footing load on a stratified soil as shown in the sketch. Statification is reasonably uniform. Allowable pressures are:

$$\begin{aligned}
\text{Stratum 1: } p_a &= 4000 \text{ psf}\\
\text{Stratum 2: } p_a &= 2000 \text{ psf}\\
\text{Stratum 4: } p_a &= 1500 \text{ psf}\\
\text{Stratum 5: } p_a &= 2000 \text{ psf}
\end{aligned}$$

To find: Whether actual pressures are within the allowable limits.



Solution:

The dispersion widths at each level are calculated.

Top of stratum 2: Width = 8 + 2 x 2/2 = 10 ft
Top of stratum 3: Width = 10 + 2 x 3/2 = 13 ft
Top of stratum 4: Width = 13 + 2 x 2/2 = 15 ft

The contact pressures are calculated:

At contact surface : p_1 = 240/(8x8) = 3750 psf
At top of stratum 2: p_2 = 240/(10x10) = 2400 psf
At top of stratum 3: p_3 = 240/(13x13) = 1420 psf
At top of stratum 4: p_4 = 240/(15x15) = 1067 psf

The foregoing results are compared with the allowable soil pressures given for the various strata. It is found that the allowable pressure on stratum 2 is exceeded; the design is therefore not acceptable.

Although corrective measures were not specifically a part of this example, such measures might be considered. One solution would be to increase the footing size to 9 feet square, thereby reducing all soil pressures, including that on stratum 2. Another solution would be to raise the founding line 1 foot, thereby increasing the dispersion width at the top of stratum 2; the extra width might reduce the pressure enough to bring it within its allowable value.

At-Rest Pressures in a Soil Mass

When pressures are uniform, there is no shear stress in the underlying soil. Until some circumstance is introduced that produces a lateral load or a discontinuity in pressure, the soil is in its state of principal stress, termed its *at-rest* condition. The vertical pressure p_1 at any point h below the surface is simply the pressure γh, where γ is the unit weight of the soil.

The unit weight of the soil can vary, however, depending on the degree of saturation. At one extreme, the unit weight is the dry unit weight γ_d, where there is no porewater at all. At the other extreme, the unit weight is the saturated unit weight γ_{SAT}, where all voids are filled with porewater. In the somewhat imaginary saturated state, all porewater is considered to be at *neutral pressure*, that is, it would exert neither negative capillary pressure on the soil grains nor would it produce buoyancy on the soil grains.

The existence of a uniform vertical pressure on the soil does, however, produce a lateral soil pressure in the stratum as well as the vertical pressure. The lateral soil pressure at any depth in the stratum can be likened to the pressures in an equivalent fluid, as shown in Fig. 5-23.

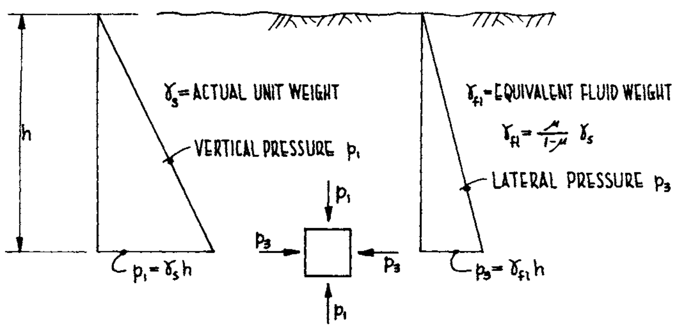


Figure 5-23 Equivalent Fluid Pressures

If the unit weight of this equivalent fluid is known, the lateral pressure at any depth is simply the hydrostatic pressure,

$$p_3 = \gamma_{EQUIV} h \tag{5-25}$$

where:

p_3 is the lateral pressure in the equivalent fluid
γ_{EQUIV} is the unit weight of the equivalent fluid
h is the standing head of the equivalent fluid

For a uniform soil mass, the theoretical unit weight g_{EQUIV} of the equivalent fluid can be computed by the methods of the theory of elasticity:

$$p_3 = \frac{\mu}{1-\mu} \gamma_{SOIL} h \tag{5-26}$$

Hence,

$$\gamma_{EQUIV} = \frac{\mu}{1-\mu} \gamma_{SOIL} \tag{5-27}$$

where: γ_{SOIL} is the actual unit weight of the soil
μ is Poisson's ratio for the soil.

For soils, the usual range of Poisson's ratio is between 0.2 and 0.4. Hence the theoretical unit weight given by Equation (5-28) should range between 25% and 67% of its actual weight. This range is not supported very well by test results, however, which indicate that a much smaller range of lateral earth pressure occurs in most soils. From experimental results, the range of values for the unit weight of the equivalent fluid has been found to range from 40% to 50% of the actual unit weight of the soil, γ_{SOIL},

$$p_3 = k \gamma_{SOIL} h \tag{5-28}$$

where $k\gamma_{SOIL}$ is the unit weight of the equivalent fluid, γ_{EQUIV}, with k between 0.4 and 0.5.

The concept of the equivalent fluid is slowly falling from general use. Nonetheless, it appears in many of the current state and federal design documents and is still in common use by older engineers. It seems likely that the concept of the equivalent fluid will be used at least for another generation.

Like other materials of construction, soils are subject to creep. As a consequence of its susceptibility to creep, soil will seek its principal state of stress under a long-term uniform load. Such behavior can produce some unwanted long-term deflections in retaining walls.

Some examples will illustrate the concepts of at-rest pressures in a soil mass, to include the effects of creep where appropriate.

Example 5-9 Lateral at-rest pressures in a soil mass

Given : Soil having a unit weight of 116 pcf used as backfill against a rigid basement wall

To Find: Lateral pressure against the wall at a depth of 6 ft below the surface after a long period of time

Solution:

When all creep has stopped, the soil will return to its principal state of stress. The lateral pressure at a depth of 6 ft. will then be:

$$p_3 = k\gamma_{SOIL}h = (0.4 \text{ to } 0.5)\times 116\times 6 = 280 \text{ to } 350 \text{ psf}$$

Example 5-10 Computation of Equivalent fluid weight

Given : Soil with unit weight of 108 pcf. Poisson's ratio = 0.28

To Find: Equivalent fluid weight of the soil

Solution:

From Equation (5-27), the theoretical equivalent fluid weight of the soil is:

$$\gamma_{EQUIV} = \frac{\mu}{1-\mu}\gamma_{SOIL} = \frac{0.28}{1-0.28}\times 108 = 42 \text{ pcf (theoretical)}$$

From experimental data, the equivalent fluid weight will probably be somewhere between 40% and 50% of its actual weight,

$$\gamma_{EQUIV} = (0.4 \text{ to } 0.5)\gamma_{SOIL} = 43 \text{ to } 54 \text{ pcf (empirical)}$$

Example 5-11 Combined lateral pressures

Given : Basement wall backfilled with a sandy soil having a saturated unit\ weight of 124 pcf.

To Find: Percent change in lateral pressure when spring flooding occurs

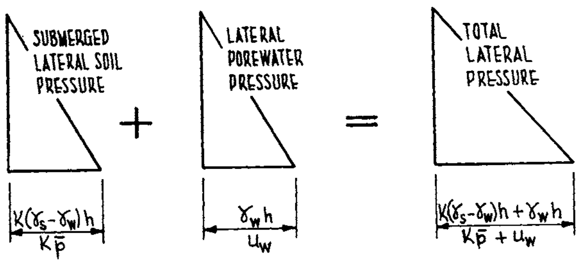
Solution:

After all movement ceases and the soil reaches its at-rest pressure (without flooding), the equivalent fluid pressure of the saturated soil will be between 40% and 50% of the soil weight of 124 pcf, or 50 to 62 pcf, for an average of 56 pcf. At this point, all voids are filled with water at neutral pressure.

When spring flooding occurs, the unit weight of the soil will be reduced by its buoyancy, becoming

$$\gamma_{SOIL} - \gamma_{WATER} = 124 - 62.4 = 61.6 \text{ pcf.}$$

The equivalent fluid weight of the soil then becomes 40% to 50% of this reduced unit weight for an average value of 28 pcf.



When spring flooding occurs, however, the excess water will develop a lateral hydrostatic pressure above the neutral pressure, thus adding to the lateral earth pressure. The total equivalent fluid weight is then 28 + 62.4 or 90 pcf. The increase in the equivalent fluid weight is then 90 - 56 or 34 pcf.

The final percentage change in lateral pressure is thus 34/56, or a 61% increase.

In Situ Properties of Soils

A particular Latin term, *in situ*, is commonly used in soil mechanics to define the in-place, undisturbed conditions in a soil mass. The term is used to describe the state of the soil, its pressures, its water content, its state of compaction and all other physical properties that existed before any man-made disturbances occurred. In this definition, the word "disturbances" also includes any disruptive influence that may have been caused by any test sampling or any other test procedures used in the soils investigation.

Such a strict description is particularly necessary when one is dealing with settlements. When a sample at some depth in a stratum of soil is removed for testing, the mere act of bringing the sample to the surface releases the at-depth pressures on the sample, both vertically and laterally. Simply restoring the in-place vertical and lateral pressures on a soil, particularly a clay soil, does not necessarily restore its original undisturbed state. Hence, its response to load when it is subjected to the test procedures becomes that of a disturbed sample.

It is necessary, therefore, to have a term that describes an entirely undisturbed state. The term *in situ* is used almost universally in soil mechanics to describe such an undisturbed and uncontaminated state of existence. Whenever the term *in situ* is used in subsequent discussions, it should always be construed to mean such a pristine state.

Review Questions

5.1 Define permeability of a soil.

5.2 Define effective pressure of a soil.

5.3 Define neutral porewater pressure in a soil.

5.4 How does submersion affect the effective pressure?

5.5 How does downward flow of a water in a soil mass affect the effective pressure? Upward flow?

5.6 How does a soil become "quick"?

5.7 What types of soil are most likely to become "quick"?

5.8 From what physical property does clay derive its strength?

5.9 What is the relationship between the unconfined compression strength of a clay and its tensile strength?

5.10 What is the relationship between the cohesive strength of a clay and its tensile strength?

5.11 Along what angle to the direction of the applied load will a sample of clay fail in the unconfined compression test?

5.12 Describe briefly the unconsolidated quick test.

5.13 Describe briefly the consolidated quick test.

5.14 Describe briefly the consolidated slow test.

5.15 What are the advantages of the vane shear test over the unconfined compression test?

5.16 Why is the direct shear test not a very satisfactory means to determine foundation conditions in sands?

5.17 What is the angle of internal friction of a sand

5.18 What happens to the shear strength of a sand supporting a footing load as the footing load is increased?

5.19 Describe briefly the SPT.

5.20 What is the Dutch cone test?

5.21 In what types of soil is a penetration test most accurate? A vane shear test?

5.22 What purpose is served in normalizing the results of a penetration test?

5.23 Physically, what does a "normalized" blow count mean? How is it different from the actual blow count obtained in the field test?

5.24 Write the Coulomb equation for shear strength in a soil and define each variable.

5.25 When submerged, which soil undergoes the higher loss in strength, sand or clay?

5.26 Under load, which settles more slowly, sand or clay?

5.27 The Boussinesq solution for pressure dispersion applies to materials that are homogeneous, elastic and isotropic. On what basis can the Boussinesq solution be applied to soils?

5.28 About what level of pressure is used for the outermost pressure contour of the Boussinesq pressure bulb when it is used to depict the pressure distribution under spread footings? Strip footings?

5.29 How far below a square spread footing of width B would the vertical pressure in the soil be reduced to about 10% of the contact pressure?

5.30 How far below a long strip footing of width B would the vertical pressure in the soil be reduced to about 15% of the contact pressure?

5.31 When the Boussinesq pressure bulb is approximated as a truncated pyramidal shape, what is the slope commonly used for the sides of the approximate bulb?

5.32 What happens to the shape of the Boussinesq pressure bulb (either actual or approximate) when the bulb extends into a second or even third stratum of different soil?

5.33 What is meant by "at-rest" pressure in a soil mass?

5.34 What is the theoretical at-rest vertical pressure at 15 ft below the surface in a soil weighing 115 pcf?

5.35 What is meant by "in situ" pressure in a soil mass?

5.36 What is meant by the "equivalent fluid weight" of a soil?

OUTSIDE PROBLEMS

5-1 A silty soil weighing 109 pcf has a field Dutch cone resistance of 60 kips/ft^2 at a depth of 10 feet. Determine the equivalent field SPT value and the normalized angle of internal friction ϕ.

5-2 What range of values of the angle of internal friction ϕ can be expected in a medium sand?

5-3 What range of values of cohesion *c* can be expected in a stiff clay?

5-4 A coarse sand weighing 119 pcf has a dutch cone resistance of 170 kips/ft^2. What is its textural classification?

5-5 A medium sand weighing 124 pcf has a field SPT blow count of 20 at a depth of 16 feet. What is its normalized blow count?

5-6 A sandy soil weighing 119 pcf has an angle of internal friction ϕ in the field of 32° at a depth of 6 feet. Determine its normalized angle of internal friction.

5-7 A clay soil has an unconfined compressive strength of 3 kips/ft^2. What is its textural classification?

5-8 A sandy soil weighing 115 pcf has a field SPT blow count of 7 at a depth of 6 feet. Determine its normalized angle of internal friction.

5-9 A deep stratum of sandy soil weighing 120 pcf has a field blow count of 24 at a depth of 20 feet. Determine the normalized angle of internal friction ϕ at that level after 8 feet of cut has been removed from the surface.

5-10 A stratum of fine sand weighing 122 pcf has a field Dutch cone resistance of 175 kips/ft^2 at a depth of 22 feet. Determine the normalized angle of internal friction ϕ of this soil.

5-11 A field SPT test in a stratum of medium clay weighing 107 pcf indicates a blow count of 7. Approximately, what would be the value of cohesion c for this clay?

5-12 What range of values of SPT blow count could be expected at a depth of 25 feet in a medium clay weighing 111 pcf?

5-13 A soil has a vane shear strength of 840 pcf and a SPT blow count of 5. Determine the theoretical value of cohesion c.

5-14 A soil has an unconfined compressive strength of 1500 psf and a Dutch cone resistance of 254 kips/ft^2. Determine its theoretical shear stress at failure.

5-15 A building 100 feet square exerts an average bearing pressure of 425 psf over its footprint area. At what depth can the increase in pressure be expected to drop to 100 psf or less?

5-16 At a depth of 12 feet below an isolated footing 6 feet square, the increase in soil pressure was measured and found to be 200 psf. Determine the approximate load being carried by the footing at the time.

5-17 A strip footing 6 feet wide has a contact pressure of 3000 psf. At what depth will the increase in soil pressure diminish to one-half the contact pressure? To one-fourth the contact pressure?

5-18 Solve problem 5-17 for a square footing 6 feet on a side and compare the results.

5-19 The backfill against a basement wall is a silty sand, classified SM, weighing 117 pcf. After a long period of time, what will be the at-rest pressure against the wall at a depth of 3 feet? At a depth of 6 feet? At a depth of 9 feet?

5-20 For the basement of problem 5-19, what will be the lateral pressure if the area becomes flooded? What is the percent increase or decrease in pressure at flood stage?

5-21 What is the equivalent fluid weight from the theory of elasticity for a soil weighing 109 pcf and having a Poisson's ratio of 0.20?

5-22 A soil has an actual unit weight of 121 pcf and an equivalent fluid weight of 59 pcf. What are the vertical and lateral pressures in this soil at a depth of 9 feet?

5-23 Determine the approximate average vertical pressure to be expected at a depth of 10 feet below a strip footing 6 feet wide having a contact pressure of 2000 psf.

5-24 Determine the approximate horizontal area of soil that will sustain a significant increase in vertical pressure at a depth of 15 feet under a footing 10 feet square carrying a load of 160 kips.

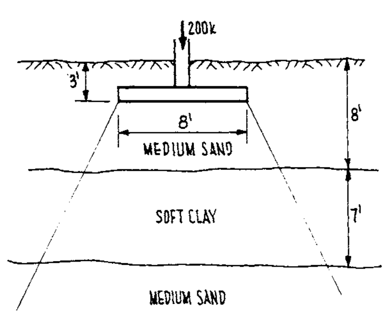
5-25 A footing is 10 feet. square. Using the Boussinesq pressure curves of Fig. 5-15, determine the pressure at the centerline of the footing at a vertical depth of 15 feet below the contact surface when the contact pressure is 1000 psf. Determine also the pressure at a distance of 12.5 feet laterally from the centerline of the footing at this same depth.

5-26 Using the approximate pressure bulbs of Fig. 5-19, determine the pressures at the points named in Problem 5-25. What is the percent error in the two sets of results?

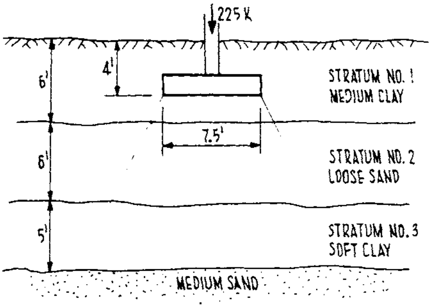
5-27 Solve Problems 5-25 and 5-26 if the footing is a strip footing.

5-28 Solve problems 5-25 and 5-26 if the vertical depth is 5 feet and the lateral distance is 7.5 feet from centerline of footing.

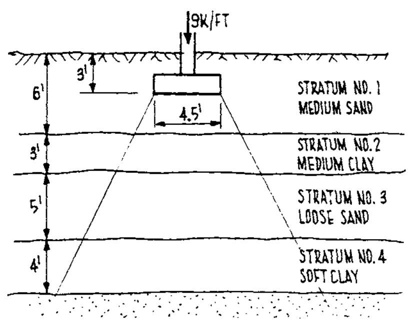
5-29 Determine the approximate average increases in pressure at the top and bottom of the indicated stratum of clay due to a footing 8 feet square founded at a depth of 3 feet below surface. Determine also the average increase in pressure at middepth of the clay stratum.



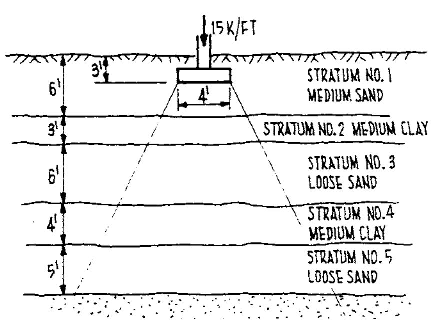
5-30 For a load of 225 kips on a footing 7.5 feet square, determine the approximate average increases in pressure at the top and bottom of each supporting stratum. Determine also the average increases in pressure at middepth of strata No. 2 and No. 3.



5-31 For a load of 9 kips/ft on a strip footing 4.5 feet wide, determine the approximate average increases in pressure at the top and bottom of each supporting stratum. Determine also the average increases in pressure at middepth of strata No.2, No.3 and No.4.



5-32 For a load of 15 kips/ft on a strip footing 5 feet wide, determine the approximate average increases in pressure at the top and bottom of each supporting stratum. Determine also the average increases in pressure at middepth of strata Nos.2, 3, 4 and 5.



Chapter 6

CALCULATION OF ALLOWABLE PRESSURES*

Levels of Accuracy of the Failure Analysis

The subject of this chapter is the determination of the allowable bearing pressures to be used in the design of shallow foundations. The discussions in this chapter are limited to those topics that are directly related to foundation design. Topics in soil mechanics other than foundation design are not included in this textbook.

As noted in earlier discussions, the number of methods used to find soil properties is quite limited, with each test being applied to a broad group of soils. Within such broad soil groups, there is inherently a wide range of the accuracy of the results, due simply to the wide variability of soil properties in each of these broad groups. As a consequence of such variability, the accuracy of calculations involving these properties is correspondingly limited.

In addition to inaccuracies in testing, inaccuracies in loading can also affect results. Placing a load on a footing creates an increase in the bearing pressure which will be dispersed downward and outward into the soil mass. At some distance downward, the load will be so widely dispersed that any further increases in soil pressure can be ignored. These increases are usually ignored when they have diminished to about 10% to 15% of the contact pressure under the footing.

Further, all such calculations are based on a soil sampling program and a field survey that gives the underground soil stratification only at the lowest possible number of selected points. Under such minimal information, the completeness of the investigation is understandably limited.

It is again emphasized that the inaccuracies just described apply to the predicted *magnitude* of the response of a soil to a load, not to the response itself. Calculations predicting the general response of a soil to a load can be treated with a degree of confidence. Calculations predicting the *magnitude* of that response are less believable.

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* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

As a very general guide, one should not expect an accuracy better than ±50% in calculations involving soils. It should be noted also that when calculations are based on a consistency or textural classification, the accuracy diminishes even more. And further, the accuracy in predicting settlements is even less than than the accuracy in predicting pressure distributions.

Ultimate Shear Failure in a Soil Mass

It has been observed that failure in soils carrying a foundation load will be due primarily to shear rather than tension or compression. A typical failure of this type is shown in Fig. 6-1, where the soil is displaced sideways along a "sheared" surface. The initial failure occurs at some angle θ which is readily determined from a Mohr's circle state of stress.

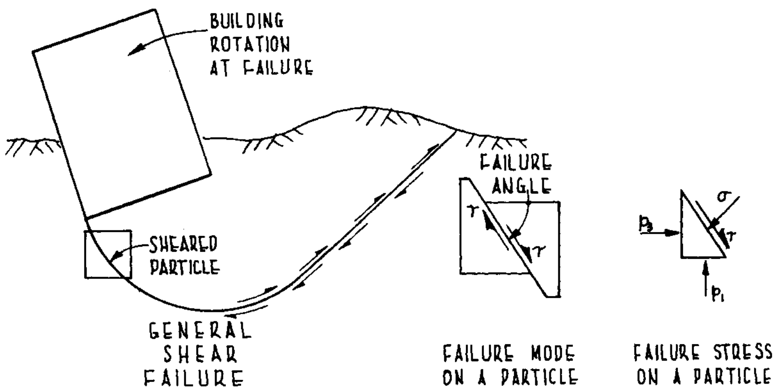


Figure 6-1 General Shear Failure in a Soil

The Coulomb equation developed in Chapter 5 provides the shear strength of a general soil having both cohesive strength from its clay component and friction strength from its sand component. A plot of the straight line representing the Coulomb equation is shown in Fig. 6-2. The angle of internal friction ϕ is also indicated, as well as the cohesion c. The Mohr's circle state of stress at failure is also shown. (Note that compression is plotted to the right.) The values of normal stresses p_1 and p_3 at failure produce a circle that is tangent to the Coulomb equation at A.

In Fig. 6-2, a line is constructed through point A, perpendicular to the straight line representing the Coulomb equation. The result is line AB, which is the diameter of the Mohr's circle that defines the state of stress at failure. Any larger circle would somewhere exceed the ultimate shear stress s' and any smaller circle would not produce a value of stress s high enough to cause failure.

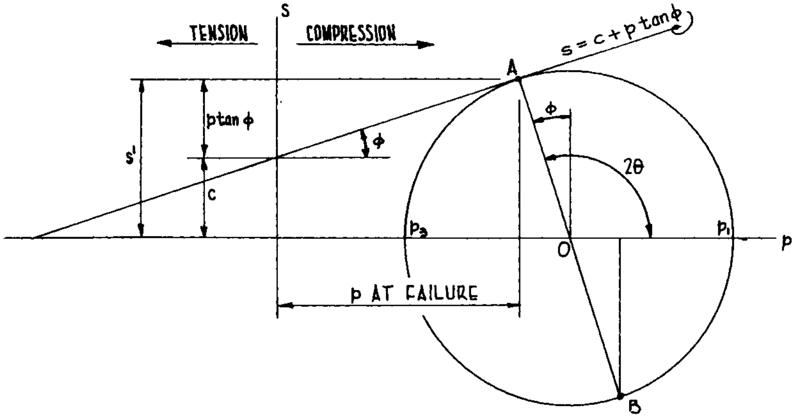


Figure 6-2 Mohr's Circle State of Stress at Failure

The angle at failure is shown as 2θ, measured from the state of initial principal stress. The value of the angle θ at failure is derived directly from the geometry,

$$2\theta = 90 + \phi \quad \text{or} \quad \theta = 45 + \frac{\phi}{2} \tag{6-1}$$

An important property of soils can be deduced from the failure angle shown in Fig. 6-2: *The failure angle θ is independent of the cohesion c.* It is concluded that a soil having both a sand component and a clay component (with no lateral load) will always fail at the same failure *angle*, $\theta = 45° + \phi/2$. The failure *strength*, however, will vary with the magnitude of the cohesion.

The location of the failure angle θ on a soil particle is shown in Fig. 6-3. Note that if a sand component exists, the failure angle will be steeper than if no sand component exists.

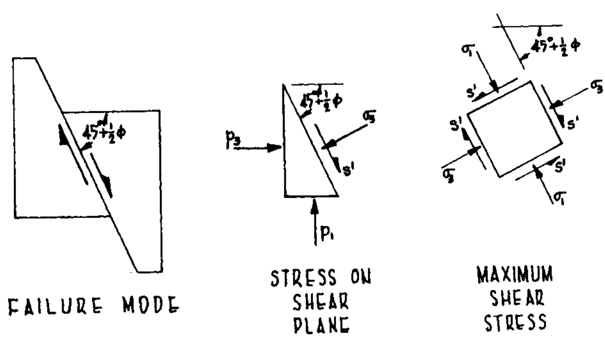


Figure 6-3 General State of Stress in a Soil at Failure

The failure angles shown in Fig. 6-3 are those that will occur when there is no lateral load. When a lateral load occurs, the failure angle can decrease markedly and can even be less than 45°. However interesting, this case of loading is not included in an elementary text such as this.

When the angle of internal friction ϕ is zero (pure clays), the state of stress becomes that of principal shear with a failure angle of 45°. Conversely, if the cohesion c is zero (pure sands), the angle of failure is $45° + \phi/2$. These two special cases are shown in Fig. 6-4; they are the limiting cases for a failure angle $45° + \phi/2$.

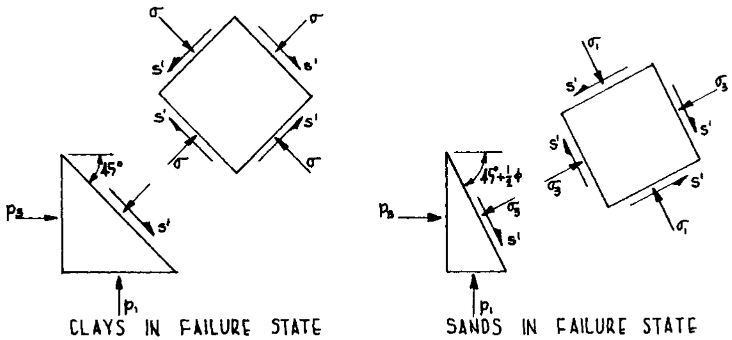


Figure 6-4 Failure States for Clays and Sands

For a soil particle at failure, the relationship between major and minor principal stresses is readily derived from the geometry of Mohr's circle. These principal stresses are indicated as p_1 and p_3 on the Mohr's circle of Fig. 6-5.

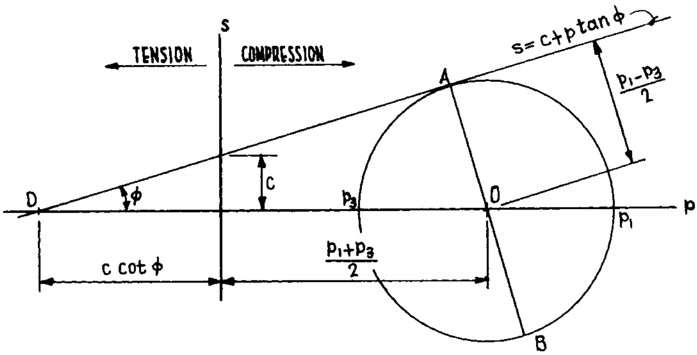


Figure 6-5 General State of Failure in a Soil

From the geometry of Fig. 6-5, a relationship involving the principal stresses p_1 and p_3 is noted in the triangle DAO.

$$\frac{p_1 - p_3}{2} = \left[\frac{p_1 + p_3}{2} + c\cot\phi\right]\sin\phi \tag{6-2a}$$

The equation is solved for the principal stress to find:

$$p_1 = p_3\frac{1+\sin\phi}{1-\sin\phi} + 2c\frac{\cos\phi}{1-\sin\phi} \tag{6-2b}$$

The following identities allow transformation of the trigonometric terms of Equation (6-2b) into the failure angle $45° + {}^{\phi}/_{2}$:

$$\frac{\cos\phi}{1-\sin\phi} = \tan(45+\frac{\phi}{2}) \qquad \frac{1+\sin\phi}{1-\sin\phi} = \tan^2(45+\frac{\phi}{2}) \tag{6-3a,b}$$

When these identities are substituted into Equation (6-2b), the relationship between principal stresses become:

$$p_1 = p_3\tan^2(45+\frac{\phi}{2}) + 2c\tan(45+\frac{\phi}{2}) \tag{6-4a}$$

or,

$$p_3 = p_1\cot^2(45+\frac{\phi}{2}) - 2c\cot(45+\frac{\phi}{2}) \tag{6-4b}$$

Equations (6-4a) and (6-4b) can be used to calculate lateral pressures in a soil at its failure state. Some examples will illustrate such calculations.

Example 6-1 Calculation of principal stress at failure

Given : A weak sand with $\phi = 29°$, $\gamma_{SOIL} = 121$ pcf, and
a strong sand with $\phi = 36°$, $\gamma_{SOIL} = 126$ pcf

To find: Ratio of lateral pressure p_3 to vertical pressure p_1 for each sand at failure load.

Solution:

The ratio of lateral pressure to vertical pressure can be determined from Equation (6-4b), where cohesion $c = 0$:

$$\frac{p_3}{p_1} = \cot^2(45+\frac{\phi}{2})$$

For the first soil, with $\phi = 29°$,

$$\frac{p_3}{p_1} = \cot^2(45+14.5) = 0.35,$$ or, p_3 is 35% of p_1 at failure

For the second soil, with $\phi = 36°$

$$\frac{p_3}{p_1} = \cot^2(45+18) = 0.26, \text{ or, } \quad p_3 \text{ is } 26\% \text{ of } p_1 \text{ at failure}$$

Example 6-2 Determination of lateral pressure at failure

Given : Spread footing on a stratum of clay 24 ft. thick, contact pressure of 4950 psf at failure.

Unconfined compression strength q_u of the clay is 1920 psf.

To find: Lateral pressure in the clay immediately below the footing when failure occurred.

Solution:

From Equation (6-4b), immediately below the footing, with $\phi = 0$ for clay, $c = {}^1/_2 q_u$ and $\cot 45° = 1$,

$$p_3 = p_1 - 2\frac{q_u}{2} = 4950 - 1920 = 3030 \text{ psf, or } 61\% \text{ of } p_1$$

Allowable Bearing Strength of a Soil Mass

The foundation bearing pressure that will cause failure in an underlying soil mass can now be derived. In the following derivation, the foundation is an infinitely long footing, representing a strip footing or a grade beam. The footing bears on a soil that has both a cohesion component and a friction component. All loads are vertical; inclined loads are deferred to a later section.

The failure mode for a bearing capacity failure is shown in the idealized curves of Fig 6-6a[12,14,39]. Failure in the soil could occur in one of three ways:

1. In shear to the right along the line BAEGI.
2. In shear to the left, along the line CADFH.
3. In "punching shear" vertically on the triangle BAC.

The single point of failure stress common to all three possibilities is point A. Point A is located at the centerline of the concentrically loaded footing at a depth of ${}^1/_2 B\tan(45° + {}^{\phi}/_2)$ below the founding line, as shown in the sketch. In all three cases, the dimension B of the strip footing is transverse to the failure surface.

A solution for the stress at point A is presented by Sowers[36], based on the approximate geometry of Fig. 6-6b. The accuracy of the Sowers approach has long been found to be within an acceptable range of accuracy for soils. Sowers' approach is followed here with only minor refinements.

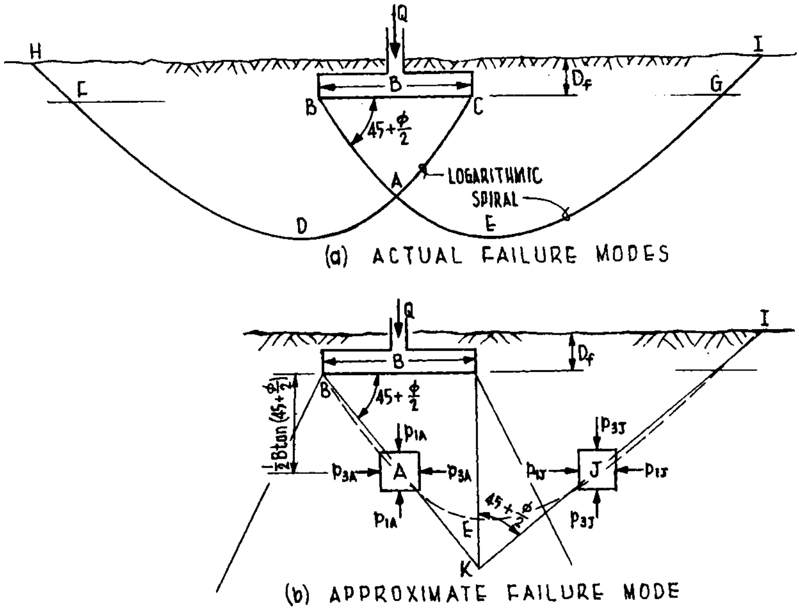


Figure 6-6 Actual and Approximate Failure Modes

The failure surface in shear to the right is taken as the failure mode for analysis. The geometry of this failure mode is shown in Fig. 6-6b. The curve BAEJI is an approximation of the actual failure curve, which is best represented as a logarithmic spiral. In the approximate curve, the angle at K is taken to be 90°, the distance AK is taken to be equal to the distance BA and the point J is located at the same depth as point A.

In Fig. 6-6a, the vertical pressure in the soil directly under the footing is known to be diminishing in accordance with the Boussinesq pressure distribution shown in Chapter 5, Figs. 5-15 and 5-19. With an approximate slope of 2 vertical to 1 horizontal for the approximate Boussinesq pressure bulb, the pressure p_1 at point A due only to footing loads is found to be:

$$p_1 = \frac{Q}{B + \frac{1}{2}B\tan(45 + \frac{\phi}{2})} = \frac{Q}{B}\left[\frac{1}{1 + \frac{1}{2}\tan(45 + \frac{\phi}{2})}\right] \tag{6-5}$$

It is recognized that $Q/B = q_0$, which is the contact pressure at the founding line at the time of failure. With this substitution, the final expression for p_1 for the vertical pressure due only to footing loads becomes

$$p_1 = q_0 \left[\frac{1}{1 + \frac{1}{2}\tan(45 + \frac{\phi}{2})} \right] \tag{6-6}$$

The total vertical pressure p_{1A} is the sum of this pressure due to footing loads plus the pressure due to overburden. With γ as the unit weight of the overburden, the total vertical pressure p_{1A} at time of failure is then:

$$p_{1A} = q_0 \left[\frac{1}{1 + \frac{1}{2}\tan(45 + \frac{\phi}{2})} \right] + \gamma D_f + \tfrac{1}{2}\gamma B \tan(45 + \frac{\phi}{2}) \tag{6-7}$$

The confining pressure at point A, p_{3A}, is given by Equation (6-4b), which for a strip footing becomes:

$$p_{3A} = q_0 \frac{1}{1 + \frac{1}{2}\tan(45 + \frac{\phi}{2})} \cot^2(45 + \frac{\phi}{2}) + \tfrac{1}{2}\gamma B \cot(45 + \frac{\phi}{2}) \\ + \gamma D_f \cot^2(45 + \frac{\phi}{2}) - 2c\cot(45 + \frac{\phi}{2}) \tag{6-8}$$

This confining pressure p_{3A} is the confining pressure at A just as failure impends. It is recognized that the horizontal pressure at point J is equal to the horizontal pressure at A . At J, however, the increase in applied pressure, p_{1J}, is now horizontal and the confining pressure, p_{3J}, is now vertical. The confining pressure p_{3J} is thus the pressure due to the weight of soil above point J,

$$p_{3J} = \gamma D_f + \tfrac{1}{2}\gamma B \tan(45 + \frac{\phi}{2}) \tag{6-9}$$

The applied pressure p_{1J} just as failure impends may be stated in terms of the confining pressure p_{3J}, again through the use of Equation (6-4b),

$$p_{1J} = p_{3J} \tan^2(45 + \frac{\phi}{2}) + 2c\tan(45 + \frac{\phi}{2}) \tag{6-10}$$

Substitution of Equation (6-9) then yields

$$p_{1J} = \gamma D_f \tan^2(45 + \frac{\phi}{2}) + \tfrac{1}{2}\gamma B \tan^3(45 + \frac{\phi}{2}) + 2c\tan(45 + \frac{\phi}{2}) \tag{6-11}$$

When the lateral pressure p_{1J} as given by Equation (6-11) is equated to p_{3A} as given by Equation (6-8), the result yields the bearing stress under a strip footing just as failure impends,

$$q_0 = \tfrac{1}{2}\gamma B[1+\tfrac{1}{2}\tan(45+\frac{\phi}{2})][\tan^5(45+\frac{\phi}{2}) - \tan(45+\frac{\phi}{2})]$$
$$+\gamma D_f[1+\tfrac{1}{2}\tan(45+\frac{\phi}{2})][\tan^4(45+\frac{\phi}{2}) - 1] \tag{6-12}$$
$$+2c[1+\tfrac{1}{2}\tan(45+\frac{\phi}{2})][\tan^3(45+\frac{\phi}{2}) + \tan(45+\frac{\phi}{2})]$$

Equation (6-12) is usually written in parametric form:

$$q_0 = \tfrac{1}{2}\gamma B N_\gamma + \gamma D_f N_q + c N_c \tag{6-13}$$

where, for a strip footing,

q_0 = contact pressure at failure

$N_\gamma = \left[1+\tfrac{1}{2}\tan(45+\frac{\phi}{2})\right]\left[\tan^5(45+\frac{\phi}{2}) - \tan(45+\frac{\phi}{2})\right]$

$N_q = \left[1+\tfrac{1}{2}\tan(45+\frac{\phi}{2})\right]\left[\tan^4(45+\frac{\phi}{2}) - 1\right]$

$N_c = \left[1+\tfrac{1}{2}\tan(45+\frac{\phi}{2})\right]\left[2[\tan^3(45+\frac{\phi}{2}) + \tan(45+\frac{\phi}{2})]\right]$

γ = effective unit weight of the soil

B = width of a strip footing (or grade beam)

D_f = depth of overburden above the founding line

The factors N_γ, Nq and N_c are called the *bearing capacity factors*. The factor N_γ reflects the effect of footing size, the factor N_q reflects the effect of depth of founding and the factor N_c reflects the effect of cohesion. There is general consensus in the practice that the form of the solution given by Equation (6-13) is correct. Discussion continues, however, over the exact form and magnitude of the bearing capacity factors themselves.

The bearing capacity factors N_γ, N_q and N_c have been under study for many years. To date, no analytical solution, including the approximation presented here, provides an entirely satisfactory mathematical solution for these factors.

Another set of the bearing capacity factors is compiled from several sources by Das[12]. These semiempirical solutions, which have evolved over a period of some 50 years, provide better correlation with experimental results than the approximate solution given by Equation (6-13), particularly at the higher and lower values of ϕ. The general form of the solution, however, remains that of Equation (6-13), but with the following improved formulas for the bearing capacity factors:

$$N_q = \tan^2(45+\frac{\phi}{2})\, e^{\pi \tan\phi}$$
$$N_c = (N_q - 1)\cot\phi \qquad (6\text{-}14a,b,c)$$
$$N_\gamma = 2\,(N_q + 1)\tan\phi$$

For the sake of comparison, the values of these two sets of factors are plotted in Fig 6-7 along with some other sets of the factors that are widely accepted in the practice[11,22,42]. Values of N_q and N_c given by Equations (6-14b and c) are shown as solid lines and the approximate values given with Equation (6-13) are shown dashed.

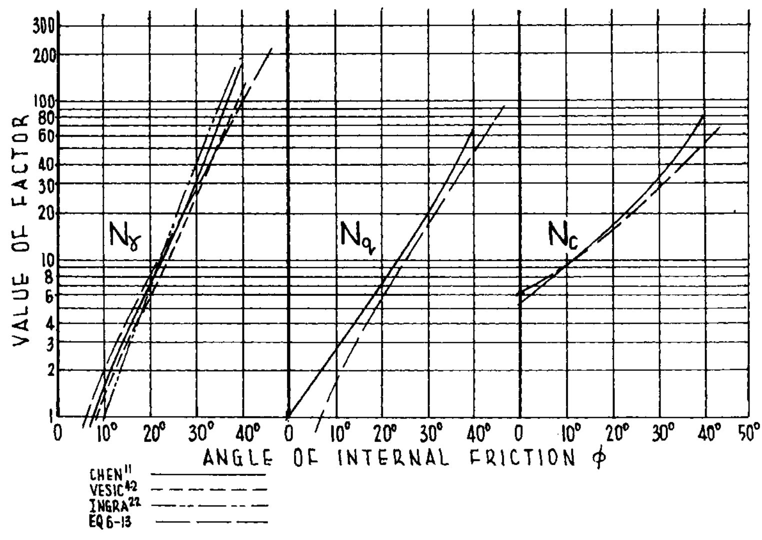


Figure 6-7 Comparison of Bearing Capacity Factors

As a matter of interest, the more accurate formulas given by Equations (6-14b and c) for N_q and N_c are generally accepted as correct. Current disagreement seems to be focused on the formula for N_γ. Obviously, the relatively close agreement shown in Fig. 6-7 indicates that any of the solutions shown there could be used safely.

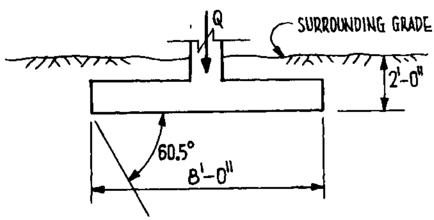
The value of q_0 given by Equation (6-13) is the contact pressure at failure, regardless which set of bearing capacity factors is used. Further, Equation (6-13) is the solution for a "standard" set of condition; it applies to a long footing that bears on a soil having both a clay component and a sand component. However, the solution does not include a factor of safety, the effects of buoyancy if flooded, the effects of lateral loads on the footing, nor the applicability of the solution to square or round footings. Suitable modifications to the solution to account for these effects are included in following sections.

Some examples will illustrate the use of Equation (6-13).

Example 6-3 Computation of bearing capacity of a soil

Given : Strip footing 8 ft wide, founding line 2 ft below the nearest free surface, founded on a stratum of sand with a unit weight of 116 pcf and an angle of internal friction of 31°.

To Find: Bearing pressure at failure



Solution:

From Equation (6-13) with c = 0 for sands and with the failure angle $\theta = 45 + \phi/2 = 60.5°$, the contact pressure at failure is computed as:

$$q_0 = \tfrac{1}{2}\,\gamma B N_\gamma + \gamma D_f N_q$$

where: $N_\gamma = [1 + \frac{1}{2}\tan 60.5][\tan^5 60.5 - \tan 60.5] = 29.2$

$N_q = [1 + \frac{1}{2}\tan 60.5][\tan^4 60.5 - 1] = 16.5$

The bearing pressure at failure is computed as:

$$q_0 = \frac{1}{2} \times 116 \times 8 \times 29.2 + 116 \times 2 \times 16.5 = 17{,}400 \text{ psf}$$

For comparison, the capacity may be found from the more accurate values of the bearing capacity factors given in Equations (6-14):

$$N_q = \tan^2(45 + \frac{\phi}{2})e^{\pi \tan \phi} = 20.631$$

$$N_\gamma = 2(N_q + 1)\tan \phi = 25.994$$

The corresponding bearing pressure at failure is:

$$q_0 = \frac{1}{2} \times 116 \times 8 \times 25.99 + 116 \times 2 \times 20.631 = 16{,}850 \text{ psf}$$

The soil will fail at a contact pressure of about 17,000 psf

Example 6-4 Computation of bearing capacity of a soil

Given : Soil and conditions of Example 6-3, footing size reduced to 3 ft wide

To Find: Bearing pressure at failure

Solution:

Refer to the sketch of Example 6-3.

For B = 3.0 ft and all other factors unchanged,

$$q_0 = \frac{1}{2} \times 116 \times 3 \times 25.99 + 116 \times 2 \times 20.63 = 9{,}310 \text{ psf}$$

The bearing strength of the soil of Example 6-8 will fail when the contact pressure reaches 9,310 psf. The strength of a sand is therefore reduced sharply when the size of the confined bearing area is reduced. In this case, the sand loses almost half its bearing strength when the footing width is reduced from 8 ft to 3 ft.

Example 6-5 Computation of the bearing capacity of a soil

Given : Conditions of Examples 6-3 and 6-4 but the supporting soil is a clay soil, $\gamma = 106$ pcf, $c = 890$ psf

To Find: Reduction in capacity when the footing width is reduced from 8 ft to 3 ft.

Solution:

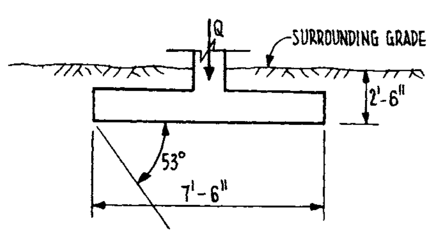
From Equation (6-13) and the bearing capacity factors given in Equations (6-14), for a clay soil having $\phi = 0$, the equation reduces to:

$$q_0 = \gamma D_f + cN_c$$

From this result, it is deduced that the bearing strength of the soil is unaffected by the size of the the footing; the strength is constant for footings of all widths.

Example 6-6 Computation of bearing capacity of a soil

Given : Strip footing 7 ft 6 in. wide founded on a soil having both a sand component and a clay component. The angle of internal friction is16° and the unconfined compressive strength is 500 psf. Unit weight is 111 pcf. Depth of founding is 2 ft 6 in. Factor of safety to bearing failure is to be 2.5.



To Find: Allowable bearing pressure under the footing

Solution:

From Equation (6-13), with $45 + \phi/2 = 53°$,

$$\tan 53° = 1.33, \qquad c = \tfrac{1}{2}q_u = 250 \text{ psf},$$

$$q_0 = \tfrac{1}{2}\gamma B N_\gamma + \gamma D_f N_q + cN_c$$

$$N_\gamma = (1 + \tfrac{1}{2} \times 1.33)(1.33^5 - 1.33) = 4.7$$

where
$$N_q = (1 + \tfrac{1}{2} \times 1.33)(1.33^4 - 1) = 3.5$$
$$N_c = (1 + \tfrac{1}{2} \times 1.33)(2)(1.33^3 + 1.33) = 12.3$$

At failure,

$$q_0 = \tfrac{1}{2} \times 111 \times 7.5 \times 4.7 + 111 \times 2.5 \times 3.5 + 250 \times 12.3$$
$$= 6000 \text{ psf}$$

With a factor of safety of 2.5,

$$\text{allowable } p_a = \frac{q_0}{2.5} = 2400 \text{ psf}$$

For comparison, the more accurate solution given by Equations (6-14) yield the following values for the bearing capacity factors:

$$N_q = \tan^2(45 + \frac{\phi}{2})e^{\pi \tan \phi} = 4.335$$
$$N_c = (N_q - 1)\cot \phi = 11.631$$
$$N_\gamma = 2(N_q + 1)\tan \phi = 3.060$$

For these bearing capacity factors, the value of q_0 is:

$$q_0 = \tfrac{1}{2} \times 111 \times 7.5 \times 3.060 + 111 \times 2.5 \times 4.335 + 250 \times 11.631$$
$$= 5384 \text{ psf}$$

With a factor of safety of 2.5,

$$\text{allowable } p_a = \frac{q_0}{2.5} = 2150 \text{ psf} \quad \text{(use)}$$

It should be apparent that having to compute the bearing capacity factors each time they are needed would indeed be a nuisance. Rather, a design aid can be prepared in terms of the angle of internal friction ϕ. Table 6-1 is one such design aid; there are others in the literature[12,14,39].

Table 6-1 Bearing Capacity Factors

$$N_q = \tan^2\left[45+\frac{\phi}{2}\right]e^{\pi\tan\phi} \qquad N_\gamma = 2(N_q+1)\tan\phi \qquad N_c = (N_q-1)\cot\phi$$

Angle of internal friction	Bearing Capacity Factors		
ϕ	N_γ	N_q	N_c
0°	0	1.000	5.142
1°	0.073	1.094	5.379
2°	0.153	1.197	5.632
3°	0.242	1.309	5.900
4°	0.340	1.433	6.185
5°	0.449	1.568	6.489
6°	0.571	1.716	6.813
7°	0.707	1.879	7.158
8°	0.860	2.058	7.527
9°	1.031	2.255	7.922
10°	1.224	2.471	8.345
11°	1.442	2.710	8.798
12°	1.689	2.974	9.285
13°	1.969	3.264	9.807
14°	2.287	3.586	10.370
15°	2.648	3.941	10.977
16°	3.060	4.335	11.631
17°	3.529	4.772	12.338
18°	4.066	5.258	13.104
19°	4.681	5.798	13.934
20°	5.386	6.399	14.835

Angle of internal friction	Bearing Capacity Factors		
ϕ	N_γ	N_q	N_c
21°	6.196	7.071	15.815
22°	7.128	7.821	16.883
23°	8.202	8.661	18.049
24°	9.442	9.603	19.324
25°	10.876	10.662	20.721
26°	12.539	11.854	22.254
27°	14.470	13.199	23.942
28°	16.717	14.720	25.803
29°	19.338	16.443	27.860
30°	22.402	18.401	30.140
31°	25.994	20.631	32.671
32°	30.215	23.177	35.490
33°	35.188	26.092	38.638
34°	41.064	29.440	42.164
35°	48.029	33.296	46.124
36°	56.311	37.752	50.585
37°	66.192	42.920	55.630
38°	78.024	48.933	61.352
39°	92.246	55.957	67 867
40°	109.411	65.195	75.313

Table 6-1 has been prepared from the more accurate values of the bearing capacity factors compiled by Das[12], presented earlier as Equations (6-14). For the sake of simplicity in subsequent discussions, only these bearing capacity factors will be used. They are not, however, the only factors being used in current practice.

As noted earlier, Equation (6-13) gives only the solution for the bearing capacity q_0 for a long strip footing at failure. This "standard" footing is subject only to vertical loads and is founded on a soil having both a clay component and a sand component. The solution so obtained, however, can be modified to suit other non-standard conditions that may occur at other sites. Such modifications involve the use of empirical correction factors which have been developed and verified over a period of years.

Five specific modifications to the bearing capacity are considered in subsequent discussions:

1. Correction of the bearing capacity factors to suit footing shapes other than long strip footings or grade beams.
2. Correction of the bearing capacity factors to reflect observed improvements in bearing capacity at increased depth of founding, both for clays and for sands.
3. Correction of the bearing capacity factors to include the effects of submersion, due either to surface flooding or to a rising water table.

4. Correction of the bearing capacity factors to reflect the loss in available strength when loads include lateral loads, such as wind and earthquake.
5. Application of a factor of safety to the corrected failure capacity that will be suitable for the specific project.

Except for the factor of safety, all of the foregoing corrections or modifications are applied to the individual bearing capacity factors, N_γ, N_q and N_c. The reason is quite simple: under some conditions, N_γ may increase in magnitude while N_q or N_c may decrease in magnitude. As a consequence, the factors are best corrected separately for specific load conditions.

Technically, it may be improper to apply the correction factors to the bearing capacity factors N_γ , N_q, and N_c, since the corrections apply to the entire term, not just to the bearing capacity factors. In order to keep track of the corrections, however, it is convenient to group the corrections and their subcripts with the factors and their like subscripts. That practice is followed in subsequent discussions.

Note that the factor of safety is applied to the bearing capacity q_0' *after all the corrections are applied.* The factor of safety therefore applies to the corrections as well as to the bearing pressure q_0.

The bearing capacity equation thus takes the final form:

$$p_a = \frac{q_0'}{FS} = \frac{1}{FS}\left[\tfrac{1}{2}\gamma B N_\gamma' + \gamma D_f N_q' + c N_c'\right] \tag{6-15}$$

where:

$$N_\gamma' = N_\gamma s_\gamma d_\gamma w_\gamma i_\gamma$$

$$N_q' = N_q s_q d_q w_q i_q$$

$$N_c' = N_c s_c d_c w_c i_c$$

and:

p_a is the allowable bearing pressure
q_0' is the modified bearing capacity at failure
FS is the factor of safety to bearing failure
N_γ, N_q, and N_c are the bearing capacity factors given by Equations (6-14a, b and c) or by Table 6-1
s_γ, s_q, and s_c are the correction factors for shape
d_γ, d_q, and d_c are the correction factors for depth
w_γ, w_q, and w_c are the correction factors for water
i_γ, i_q, and i_c are the correction factors for inclined loads (wind and earthquake)

Corrections for Shape of Footings

When the footing is not long and continuous as assumed in the derivation of Equation (6-13), corrections must be applied to compensate for end effects. (It is emphasized that these corrections have to do with footing *shape*, not footing *size*.) The correction factors are denoted s_γ, s_q and s_c in Equation (6-15). Several sets of correction factors for various lengths and shapes have been developed.

Currently, one of the more popular sets of correction factors are those developed by Brinch Hansen[7,8]. For rectangular footings of width B and length L, $L > B$, the correction factors are:

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

$$s_q = 1 + \frac{B}{L}\tan\phi \qquad \text{(6-16a, b, c)}$$

$$s_c = 1 + \frac{B}{L} \times \frac{N_q}{N_c}$$

where ϕ is the angle of internal friction and N_q and N_c are the uncorrected bearing capacity factors given by Equations (6-14b and c) or by Table 6-1.

Table 6-2 Correction Multipliers[a] to the Bearing Capacity Factors Due to Shape[b,c] of Footing

$\frac{B}{L}$	Multiplier	Angle of internal friction ϕ										
		0°	4°	8°	12°	16°	20°	24°	28°	32°	36°	40°
0	s_γ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	s_q	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	s_c	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.2	s_γ	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92	0.92
	s_q	1.00	1.01	1.03	1.04	1.06	1.07	1.09	1.11	1.12	1.15	1.17
	s_c	1.04	1.04	1.05	1.06	1.07	1.09	1.10	1.11	1.13	1.15	1.17
0.4	s_γ	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84	0.84
	s_q	1.00	1.03	1.06	1.09	1.11	1.15	1.18	1.21	1.25	1.29	1.34
	s_c	1.08	1.09	1.11	1.13	1.15	1.17	1.20	1.23	1.26	1.30	1.34
0.6	s_γ	0.76	0.76	0.76	0.76	0.76	0.76	0.76	0.76	0.76	0.76	0.76
	s_q	1.00	1.04	1.08	1.13	1.17	1.22	1.27	1.32	1.37	1.44	1.50
	s_c	1.12	1.14	1.16	1.19	1.22	1.26	1.30	1.34	1.39	1.45	1.51
0.8	s_γ	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68	0.68
	s_q	1.00	1.06	1.11	1.17	1.23	1.29	1.36	1.43	1.50	1.58	1.67
	s_c	1.16	1.19	1.22	1.26	1.30	1.34	1.40	1.46	1.52	1.60	1.68
1.0	s_γ	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60	0.60
	s_q	1.00	1.07	1.14	1.21	1.29	1.36	1.45	1.53	1.62	1.73	1.84
	s_c	1.19	1.23	1.27	1.32	1.37	1.43	1.50	1.57	1.65	1.75	1.85

[a]Brinch Hansen[7,8]
[b]Rectangular footings of width B and length L
[c]For circular shapes, use B/L = 1

As one might imagine, it would be a nuisance to compute the correction factors each time they are needed. A design aid listing the values of the correction factors for various values of B/L and ϕ is indicated. Such a design aid has been prepared aside and is presented as Table 6-2.

Corrections for Depth of Founding

In the derivation of the bearing capacity q_0, the effects of the depth of founding D_f were included, appearing as the second term in Equation (6-13). However, the resulting values of q_0 do not agree well with experimental observations concerning depth of founding. The most significant observation is that the strength of clay soils, like sands, increases as the depth of founding increases. A correction factor is therefore applied to improve the computed value of q_0. The correction factors are denoted d_γ, d_q and d_c in Equation (6-15).

One of the empirically developed sets of correction factors is that proposed by Brinch Hansen[7,8] and Vesic[43]. A table of the Brinch Hansen factors is given in Table 6-3, following.

Table 6-3 Correction Multipliers[a] to the Bearing Capacity Factors Due to Depth of Founding[b]

$\frac{D_f}{B}$	Multi-plier	Angle of internal friction ϕ										
		0°	4°	8°	12°	16°	20°	24°	28°	32°	36°	40°
0	d_γ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	d_q	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	d_c	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
0.25	d_γ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	d_q	1.00	1.03	1.05	1.07	1.08	1.08	1.08	1.08	1.07	1.06	1.05
	d_c	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
0.50	d_γ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	d_q	1.00	1.06	1.10	1.13	1.15	1.16	1.16	1.15	1.14	1.12	1.10
	d_c	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20	1.20
0.75	d_γ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	d_q	1.00	1.09	1.16	1.20	1.23	1.24	1.24	1.22	1.21	1.19	1.16
	d_c	1.30	1.30	1.30	1.30	1.30	1.30	1.30	1.30	1.30	1.30	1.30
1.00	d_γ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	d_q	1.00	1.12	1.21	1.27	1.30	1.32	1.31	1.30	1.28	1.25	1.21
	d_c	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40
2.00	d_γ	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	d_q	1.00	1.13	1.23	1.30	1.33	1.35	1.35	1.33	1.31	1.27	1.24
	d_c	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44	1.44

[a]Brinch Hansen[7,8] and Vesic[43]
[b]Rectangular footings having depth of founding D_f and width B

The Brinch Hansen correction factors yield a modest improvement in the bearing capacity, with the most notable effects being in clay soils. In mathematical form, the Brinch Hansen correction factors are, with the factor $tan^{-1}(D_f/B)$ in radians:

For $D_f/B \leq 1.0$:

$$d_\gamma = 1.0$$

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 \frac{D_f}{B}$$

$$d_c = 1 + 0.4\frac{D_f}{B}$$

(6-17a, b, c)

For $D_f/B > 1.0$:

$$d_\gamma = 1.0$$

$$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 \tan^{-1}\left(\frac{D_f}{B}\right)$$

$$d_c = 1 + 0.4\tan^{-1}\left(\frac{D_f}{B}\right)$$

(6-17d, e, f)

Corrections for Groundwater Level

It has been noted repeatedly that the strength of a sand is directly proportional to the unit weight γ. When something occurs to decrease the unit weight of the sand, such as submersion, the strength of the sand decreases correspondingly. Unlike clays, sands are severely affected by spring flooding or by the location of the water table. A typical case of a rising water table intruding into the Boussinesq pressure bulb is shown in Fig. 6-8.

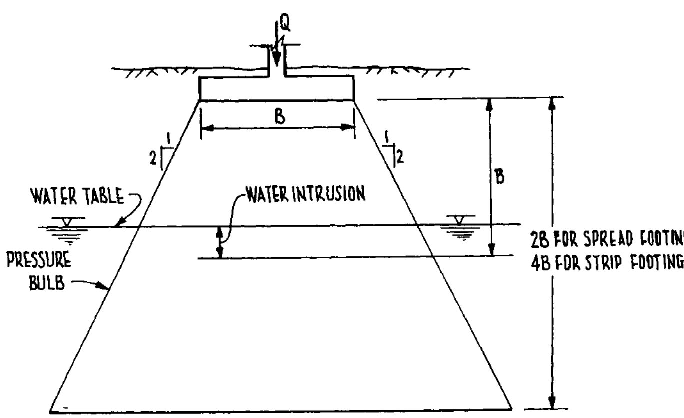


Figure 6-8 Partial Flooding under a Loaded Footing

A typical sand will weigh about 125 pcf. When submerged, the intergranular pressure of the soil, γ_{SOIL}, will be decreased by the buoyancy of the water, γ_{WATER}. The intergranular pressure is then

$$\gamma_{SOIL} - \gamma_{WATER} = 125 - 62.4 = 62.6 \text{ pcf.}$$

With this loss of roughly half the intergranular pressure, there is a corresponding loss of half the strength. For the sake of design, *it is commonly assumed that sand will lose half its strength when submerged.*

In foundation design, it is not necessary that actual flooding occur, only that the water under the footing rises to within a depth B below the founding line, as indicated in Fig.6-8. At that point, the strength of all the sand in the Boussinesq pressure bulb is considered to be submerged and has therefore lost half its strength. There is little to be gained in considering partial submergence; if partial submergence can occur, it may well be assumed that full submergence can occur.

The correction factors for water intrusion are denoted w_γ, w_q and w_c in Equation (6-15). Technically, the factor should be applied to the unit weight γ, but as noted earlier, the same end result can be obtained by applying the correction to the bearing capacity factors. Table 6-4 presents the correction factors in tabular form.

Table 6-4 Correction Multipliers to the Bearing Capacity Factors Due to Water Intrusion[a,b]

Intrusion Conditions	Correction Factors w_γ	w_q	w_c
With no Intrusion	1.00	1.00	1.00
With Intrusion	0.50	0.50	1.00

[a]Water intrusion occurs when the water table rises to within a depth B of the founding line or when the area becomes flooded for any reason.
[b]The dimension B is the least dimension of a rectangular footing or the diameter of a circular footing.

As indicated in Table 6-4, the corrections for water intrusion will apply only to the factors that have the unit weight γ in the term. In Equation (6-15), these are the first and second terms.

Note also in Table 6-4 that the factor w_c is always 1.00 for clay soils. Clays are relatively unaffected by short-term submergence. Only sands are affected by submergence.

Corrections for Lateral Loads

Lateral loads on a building must eventually be transmitted to the soil by one or more of the footings. Such a load case is shown in Fig. 6-9, where a shearing force that was never considered in earlier discussions is introduced into the soil. The overall effects of such combined loads can be a serious concern.

The state of stress illustrated in Fig. 6-9 is shown first for the usual state of principal stress, then for the state of stress when a transverse lateral stress is suddenly added. The difference in the two states of stress is that the available strength of the soil is suddenly reduced when the failure angle is introduced.

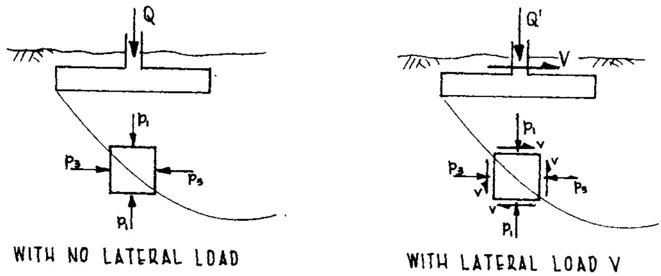


Figure 6-9 Addition of a Lateral Load to a Footing

The transverse lateral load V is transmitted to the soil by friction. Consequently, the maximum lateral force V that can be developed is the coefficient of friction μ times the vertical load Q that exists at the time,

$$\text{maximum } V = \mu Q \tag{6-18}$$

For soils, the coefficient of friction is taken to be about 0.3. For values of $V/Q > 0.3$, the footing will slide.

When wind loads act on a foundation, the base shear V is determined by the methods of Chapter 3. In those calculations, the base shear and overturning moment are calculated using the lateral wind forces acting against the outside of the structure. Vertical loads on individual footings do not enter the calculations.

When earthquake loads act on a foundation, the base shear V is also determined by the methods of Chapter 3. In the earthquake calculations, however, the vertical dead weight W undergoing acceleration is used to determine the base shear V. It should be noted that this vertical dead weight W is not the total load that actually exists on the foundations at the time the earthquake occurs; it is only the portion of the total load that is undergoing accelerations.

The ratio V/Q for an individual footing is the tangent of the angle of inclination α of the sustained load, as shown in Fig.6-10. The ratio V/Q is computed for both wind and earthquake using the "most probable" sustained load Q on the footing. The most probable sustained load that will exist throughout the day-to-day service life of a structure is commonly taken to be DL + 50%LL.

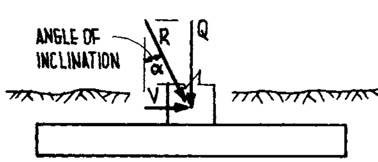


Figure 6-10 Inclined Resultant Force on a Footing

The soil under the footing undergoes a sharp reduction in capacity due to the introduction of the lateral shears and the resultant combined stresses. As yet, there is no accurate comprehensive solution to predict the loss in capacity. In practice, tables or graphs derived from experimental data are commonly used for design.

The parameter V/Q is used to develop correction factors for the bearing capacity factors when lateral loads occur. The correction factors are denoted i_γ, i_q and i_c in Equation (6-15). The choice of i for the symbol reflects the fact that the resultant load is an *inclined* load.

Several sets of correction factors can be found in the literature. One of more widely used sets is one developed by Meyerhof[26]. Meyerhof uses the angle of inclination of load α as one parameter, as shown above in Fig. 6-10.

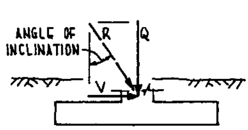


Table 6-5 Correction Multipliers[a] to the Bearing Capacity Factors Due to Inclination of Load

$\frac{V}{Q}$	i_γ											i_q i_c
	Angle of internal friction ϕ											
	0°	4°	8°	12°	16°	20°	24°	28°	32°	36°	40°	
0.02	0	0.51	0.73	0.82	0.86	0.89	0.91	0.92	0.93	0.94	0.94	0.97
0.04	0	0.18	0.51	0.65	0.73	0.78	0.82	0.28	0.86	0.88	0.89	0.95
0.06	0	0.02	0.33	0.51	0.62	0.69	0.73	0.77	0.80	0.82	0.84	0.93
0.08	0	0	0.18	0.38	0.51	0.59	0.66	0.70	0.73	0.76	0.78	0.90
0.10	0	0	0.08	0.27	0.41	0.51	0.58	0.63	0.67	0.71	0.73	0.88
0.12	0	0	0.02	0.18	0.33	0.43	0.51	0.57	0.62	0.66	0.69	0.85
0.14	0	0	0	0.11	0.25	0.36	0.45	0.51	0.56	0.61	0.64	0.83
0.16	0	0	0	0.06	0.19	0.30	0.39	0.46	0.51	0.56	0.60	0.81
0.18	0	0	0	0.02	0.13	0.24	0.33	0.40	0.46	0.51	0.55	0.79
0.20	0	0	0	0	0.09	0.19	0.28	0.36	0.41	0.47	0.51	0.76
0.22	0	0	0	0	0.05	0.14	0.23	0.31	0.37	0.43	0.48	0.74
0.24	0	0	0	0	0.02	0.11	0.19	0.27	0.33	0.39	0.44	0.72
0.26	0	0	0	0	0.01	0.07	0.15	0.23	0.30	0.35	0.40	0.70
0.28	0	0	0	0	0	0.05	0.12	0.19	0.26	0.32	0.37	0.68
0.30	0	0	0	0	0	0.03	0.09	0.16	0.23	0.29	0.34	0.66

[a]Meyerhof[25]
V = Base shear, Q = DL + 50%LL for wind, Q = DL only for earthquake

With the angle of internal friction ϕ and the angle of inclination α in degrees, Meyerhof's correction factors are given by:

$$i_\gamma = (1 - \frac{\alpha}{\phi})^2$$

$$i_q = (1 - \frac{\alpha}{90^\circ})^2 \qquad (6\text{-}20a,b,c)$$

$$i_c = (1 - \frac{\alpha}{90^\circ})^2$$

It is observed that rotational moment on the footing is not included in Meyerhof's factors. The factors are therefore most accurate where the footings are hinged to the structure above.

The usual design aid listing Meyerhof's factors is that given above as Table 6-5. For convenience, the values are listed directly in terms of V/Q and ϕ, obviating any need for finding the angle of inclination α.

Common factors of Safety in Soils

Unlike other construction materials such as steel or concrete, there is no design code specifying the allowable stresses or factors of safety to be used for design in soils. In steel beams, for example, the allowable stress is specified by the AISC design specification as $0.60F_y$, where F_y is the yield stress. The specified allowable stress thus provides a factor of safety of 1.67 to yield.

Soil is far too variable for such a specific factor of safety to be applied as a general requirement. Rather, the factor of safety for a soil is selected by the engineer for each project, based on the reliabilty of information on the loads as well as on the soil. Both the certainty of loading and the believability of the soils information must enter the selection of the factor of safety[14].

As indicated in Equation (6-15), the allowable bearing pressure p_a is found by dividing the bearing capacity at failure q_0' by a suitable factor of safety. What constitutes "suitable" is left to the engineer. A general guide for selecting the factor of safety is presented in Table 6-6. While the considerations listed in the Table 6-6 are the more common ones, the list is by no means a complete one.

The factor of safety is applied to the bearing capacity at failure. As such, it applies only to the strength of the soil. The other major consideration, settlement, has yet to be considered.

Table 6-6 Guidelines for Choosing a Factor of Safety in Soils

Type of Structure or Project	Frequency of Loading to Maximum Levels	Completeness and/or Reliability of Soils Info.	Common Factor of Safety
Monumental, long-life prestigious buildings or structures, publicly or privately owned. Expensive, first-class construction.	Rarely loaded to maximum levels but likely to be remodeled several times during its long service life. Consequences of failure serious.	Complete and reliable soils information:	3.0
		Marginal soils information:	4.0
Service structures. Bridges, retaining walls, basements, and drainage structures.	High levels of sustained load throughout life of structure. Occasionally loaded to maximum load. Consequences of failure disastrous.	Complete and reliable soils information:	3.0
		Marginal soils information:	4.0
Routine commercial or industrial property, 30-year service life, average quality of construction	Maximum design load could occur occasionally. Sustained load probably DL+50%LL. Consequences of failure serious.	Complete and reliable soils information:	2.5
		Marginal soils information:	3.5
Apartments, office buildings, light commercial structures. Primary purpose is to provide for human occupancy or work.	Maximum design load unlikely to occur. Failure most likely to be gradual and visibly apparent, with no sudden collapse.	Complete and reliable soils information:	2.0
		Marginal soils information:	3.0
Building contractor's ancillary structures. Temporary facilities for any purpose, but subject to continual use and observation.	Maximum design load likely to occur regularly. Failure likely to be progressive but subject to remedial repairs over the short service life.	Complete and reliable soils information:	1.5
		Marginal soils information:	2.5

Use of a Reference Footing in Strength Calculations

When Equation (6-15) is used in design, it is customary to ignore the cohesion c in soils classified as gravels, sands or low plasticity silts (ML). Similarly, it is customary to take the angle of internal friction ϕ to be essentially zero in soils classified as clays or high plasticity silts (MH). The reliability of a friction component in a clay or a cohesion component in a sand is sometimes questionable; they are usually ignored.

An examination of Equation (6-15) reveals that the footing width B must be known if a calculation of the allowable pressures p_a and p_a' is to be made. The rather annoying problem thus arises that the pressure cannot be found until the width B is known and the width B cannot be found until the pressure is known. *Every footing size in a group of footings will have a different allowable bearing pressure.* Such a feature can make footing design a very tedious trial-and-correction procedure.

For design of an individual footing in a group of footings whose sizes are yet unknown, the use of a nonexistent reference footing of some arbitrarily selected size provides a useful labor-saving approach. The allowable pressures p_a, p_a' and p_a'' are determined for this reference footing under the same load conditions and soil conditions as the actual footings. It will be shown in subsequent discussions that the sizes and settlements of all other like footings in the group can then be determined by a simple comparison to the size and settlement of the reference footing.

In most structures, there will be more than one set of load conditions in a group of footings. One example of such multiple load conditions would be the shearwall footings and non-shearwall footings in a braced frame. Another example would be the perimeter footings and the interior footings in a rigid frame. For such cases of plural load conditions, a separate reference footing should be established for each subset of load conditions.

In making comparisons to a reference footing, it has been found more convenient to make the comparisons using design loads rather than design pressures. For two footings on sand, for example, the maximum allowable pressure as limited by the strength of the soil is given by Equation (6-15) as:

$$p_a \ or \ p_a' = \frac{1}{FS}\left[\tfrac{1}{2}\gamma B N_\gamma' + \gamma N_q'\right] = \frac{\gamma B}{FS}\left[\tfrac{1}{2}N_\gamma' + \frac{D_f}{B}N_q'\right] \qquad (6\text{-}21)$$

When two footings in a group of footings are compared, the footings will likely be at the same depth D_f and will likely have widths B in the same general range of sizes. Consequently, comparing one footing in the group to a reference footing using Equation (6-21) yields the following ratio:

$$\frac{p_{a1}}{p_{a(ref)}} = \frac{B_1}{B_{ref}} \quad \text{or} \quad \frac{p'_{a1}}{p'_{a(ref)}} = \frac{B_1}{B_{ref}} \tag{6-22}$$

To express the ratio in terms of load rather than pressure, the equation is multiplied through by $B_1{}^2/B_{ref}{}^2$ to find:

$$\text{At maximum strength of a sand,} \quad \frac{P_1}{P_{ref}} = \frac{B_1^3}{B_{ref}^3} \tag{6-23}$$

Similarly for footings on clay, the maximum allowable pressure is also given by Equation (6-15) where, with $\phi = 0$, $N\gamma = 0$ and:

$$p_a \ or \ p'_a = \frac{1}{FS}\left[\gamma D_f N'_q + cN'_c\right] \tag{6-24}$$

When two footings on clay are compared,

$$\frac{p_{a1}}{p_{a(ref)}} = 1 \quad \text{or} \quad \frac{p'_{a1}}{p'_{a(ref)}} = 1 \tag{6-25}$$

To express the ratio in terms of load rather than pressure, the equation is multiplied through by $B_1{}^2/B_{ref}{}^2$ to find:

$$\text{At maximum strength of a clay,} \quad \frac{P_1}{P_{ref}} = \frac{B_1^2}{B_{ref}^2} \tag{6-26}$$

The foregoing relationships are summarized:

- Comparison of footing i to a reference footing on sand:

  If the two footings are designed at the maximum allowable strength of a sand, the footing loads will be proportional to the cube of the widths B,

$$\frac{P_i}{P_{ref}} = \frac{B_i^3}{B_{ref}^3} \tag{6-23}$$

- Comparison of footing i to a reference footing on clay:

  If the two footings are designed at the maximum allowable strength of a clay, the footing loads P will be proportional to the square of the widths B,

$$\frac{P_i}{P_{ref}} = \frac{B_i^2}{B_{ref}^2} \tag{6-26}$$

Since the pressures p_a, p_a' and p_a'' and size B of the reference footing will be known, the load P_{ref} on the reference footing for each of the three pressures is readily computed as $P_{REF} = pB_{REF}{}^2$. The required size of any other like footing in the group of footings is then found simply by evaluating Equations (6-23 and

(6-26). This concept of a reference footing having an arbitrarily selected size is also extended into the settlement calculations in Chapter 8.

The size selected for the reference footing is quite arbitrary. In general, a width B of 6 or 8 feet is usually a convenient size for buildings up to about 3 stories. For higher buildings, a width of 10 or 12 feet might be used. For footings at the end of a shear panel, a width of 12 to 16 feet should be expected. Since the reference footing is nonexistent, any size within the anticipated range of sizes will do.

Applications in Calculating Bearing Capacity

Some examples will illustrate the application of the allowable bearing capacity equation, Equation (6-15), in establishing the allowable pressure on the soil supporting a shallow foundation. It is well to note again that the allowable pressure obtained from Equation (6-15) is based only on the failure strength of the soil. Settlements have never been considered.

It is worth repeating that two completely independent calculations are needed when lateral loads occur on a foundation: 1) base shear and overturning moment as external actions on the foundation, and 2) frictional resistance and restoring moment produced by the foundation as reactions to the external load.

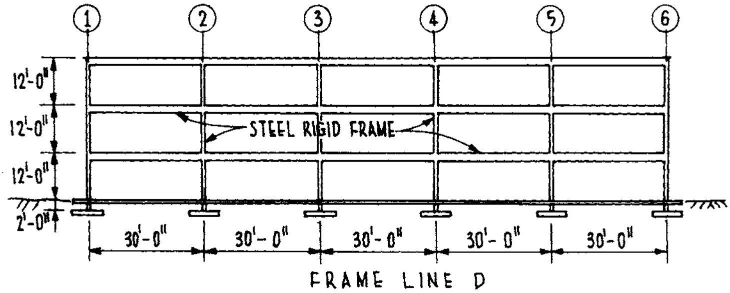
Example 6-7 Allowable bearing pressures p_a and p_a' for an interior footing in a rigid frame line

Given : Interior footing in a three story steel rigid frame apartment building. A typical frame line, labeled line D, is shown in the sketch.

At interior footing D2, dead load is 70 kips, live load is 50 kips. Total base shear on frame line D is 55 kips due to wind and 60 kips (at elastic levels) due to earthquake.

Center of lateral loads h_{LAT} is 21.9 ft above top of footings for wind and 26.6 ft for earthquake.

All columns are hinged to their footings.



A reference footing 6 ft square is selected, founded at a depth of 3 ft below grade.

The soil is classified SP, having a SPT blow count of 13, normalized to a pressure of 1500 psf. Cohesion c is essentially zero, $\gamma = 121$ pcf. The water table can rise to within 5 ft. of the surface.

To find: Allowable bearing pressures p_a and p_a' of the reference footing under the load and soil conditions at the interior footings

Solution:

The maximum allowable bearing pressure p_a at the site is given by Equation 6-15, with $c = 0$.

$$p_a = \frac{1}{FS}\left[\tfrac{1}{2}\gamma B N_\gamma' + \gamma D_f N_q'\right]$$

For a SPT blow count of 13, the angle of internal friction is found from Fig. 5-11, $\phi = 31°$.

Bearing capacity factors are found in Table 6-1:

$$N_\gamma = 25.99 \qquad N_q = 20.63$$

Correction factors for shape are found by interpolation in Table 6-2, with B/L = 6/6 =1.0:

$$s_\gamma = 0.60 \qquad s_q = 1.60$$

Correction factors for depth of burial are found in Table 6-3 for $D_f B = 3/6 = 0.5$

$$d_\gamma = 1.00 \qquad d_q = 1.14$$

Correction factors for water intrusion are found in Table 6-4

$$w_\gamma = 0.50 \qquad w_q = 0.50$$

For a commercial apartment building, an average factor of safety is found in Table 6-6 to be 2.5.

The allowable bearing pressure p_a for a reference footing size of 6 ft. square at a depth of 3 ft. is computed as:

$$p_a = \frac{1}{2.5}\left[\tfrac{1}{2} \times 121 \times 6 \times 25.99 \times 0.60 \times 1.00 \times 0.50 + 121 \times 3 \times 20.63 \times 1.60 \times 1.14 \times 0.50\right]$$

$$= 3860 \text{ psf}$$

With wind load added, the allowable bearing pressure p_a' is the same as p_a but with additional corrections for inclined load. The probable load Q is assumed to be DL + 50%LL. For each bay, $V = 55/5 = 11$ kips.

$$\frac{V}{Q} = \frac{11}{70 + 25} = 0.12$$

Correction factors for inclined load are found in Table 6-5 with $\phi = 31°$:

$$i_\gamma = 0.61 \qquad i_q = 0.85$$

The allowable bearing pressure p_a' for the reference footing size of 6 ft. square at a depth of 3 ft. is computed as:

$$p_a' = \frac{1}{2.5}\Big[\tfrac{1}{2}\times 121\times 6\times 25.99\times 0.60\times 1.00\times 0.50\times 0.61$$

$$+121\times 3\times 20.63\times 1.60\times 1.14\times 0.50\times 0.85\Big]$$

$$= 3010 \text{ psf (for wind loads)}$$

With earthquake load added, the allowable bearing pressure p_a' is the same as p_a but with further corrections added to account for the inclined load. For earthquake, the sustained load Q is again taken to be (DL + 50%LL). The lateral load V is 60/5 or 12 k.

$$\frac{V}{Q} = \frac{12}{70+25} = 0.13$$

Correction factors for inclined load are found in Table 6-5 for $\phi = 31°$,

$$i_\gamma = 0.58 \qquad\qquad i_q = 0.84$$

The allowable bearing pressure p_a' for a reference size of 6 ft. square at 3 ft. burial is computed as:

$$p_a' = \frac{1}{2.5}\Big[\tfrac{1}{2}\times 121\times 6\times 25.99\times 0.60\times 1.00\times 0.50\times 0.58$$

$$+121\times 3\times 20.63\times 1.60\times 1.14\times 0.50\times 0.84\Big]$$

$$= 2950 \text{ psf (for earthquake loads)}$$

1) The pressures to be carried forward into the design of all interior footings in the frame line are those for the reference footing (see Example 9-1):

$p_a = 3860$ psf, $p_a' = 3010$ psf with wind load,
$p_a' = 2950$ psf with earthquake load

Example 6-8 Continuation of Example 6-7; allowable bearing pressures p_a and p_a' for an end footing in a rigid frame line

Given : Typical end footing in the frame line of the apartment building of Example 6-7.

Assume the reference footing to be the same size and depth as the interior footings.

For the footing at grid line D1, DL = 55 kips, LL = 40 kips

Assume all columns are hinged to their footings

To find: Allowable bearing pressures p_a and p_a' for the reference footing under the load and soil conditions at the end footings

Solution:

Since the size of the reference footing is unchanged, the maximum allowable bearing pressure p_a for maximum gravity loads is unchanged from the solution of Example 6-7.

$$p_a = 3860 \text{ psf}$$

With wind load added, the allowable pressure p_a' with both normal and shear stress is the same as p_a but with added corrections for inclined load. For wind, the probable load Q is taken to be DL + 50%LL. For the end footing, $V = {}^1/_2(55/5) = 5.5$ kips. Hence,

$$\frac{V}{Q} = \frac{5.5}{55+20} = 0.07$$

Correction factors for inclined load are found in Table 6-5 for $\phi = 31°$

$$i_\gamma = 0.75 \qquad\qquad i_q = 0.91$$

The allowable bearing pressure for a reference footing size of 6 ft.square with burial depth of 3 ft. is now computed (see Example 6-7 for the form of the computation for p_a'):

$$p_a' = \frac{1}{2.5}\Big[\tfrac{1}{2} \times 121 \times 6 \times 25.99 \times 0.60 \times 1.00 \times 0.50 \times 0.75$$

$$+ 121 \times 3 \times 20.63 \times 1.60 \times 1.14 \times 0.50 \times 0.91\Big]$$

$$p_a' = 3335 \text{ psf (for wind loads)}$$

With earthquake load added, the allowable pressure p_a' with both normal and shear stress is the same as p_a but with added corrections for inclined load. For earthquake, the load Q is again DL + 50%LL. For the end bay, $V = {}^1/_2(60/5) = 6$ kips.

$$\frac{V}{Q} = \frac{6}{55+20} = 0.08$$

Correction factors for inclined load are found in Table 6-5 for $\phi = 31°$.

$$i_\gamma = 0.72 \qquad\qquad i_q = 0.90$$

The allowable bearing pressure p_a' for a reference size of 6 ft. square with a burial depth of 3 ft. is now computed (see Example 6-7 for the form of the computation for p_a):

$$p_a' = \frac{1}{2.5}\Big[\tfrac{1}{2} \times 121 \times 6 \times 25.99 \times 0.60 \times 1.00 \times 0.50 \times 0.72$$

$$+ 121 \times 3 \times 20.63 \times 1.60 \times 1.14 \times 0.50 \times 0.90\Big]$$

$$p_a' = 3270 \text{ psf (for earthquake loads)}$$

The pressures to be carried forward into the design of all end footings in the frame line are those for the reference footing (see Example 9-2):

$p_a = 3860$ psf, $\qquad p_a' = 3335$ psf with wind loads

$\qquad\qquad\qquad\quad p_a' = 3270$ psf with earthquake loads

Example 6-9 Allowable bearing pressures p_a and p_a' for shearwall foundations in a braced frameline

Given : Commercial diaphragm-shearwall concrete office building of Chapter 2, Fig. 2-1.

Footings A1 and A2 as shown in the frameline.

A reference footing 14 ft. square is selected, depth of founding 4 ft.

From the soils report, the soil is CH, s = 1250 psf, ϕ is essentially zero. Unit weight γ is 106 pcf. Occasional spring flooding occurs.

Footing loads are given Example 3-4:

Footing A1, DL = 183 kips, LL = 24 kips

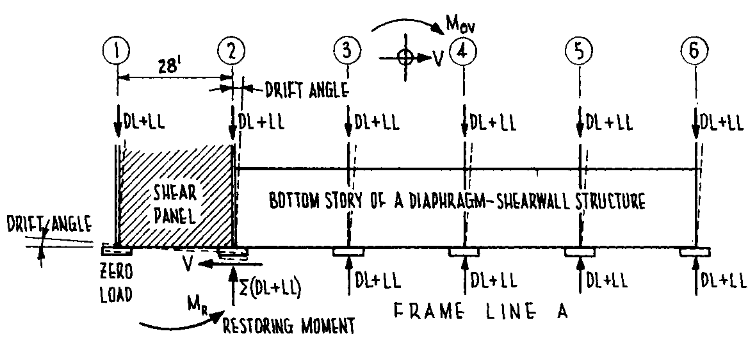
Footing A2, DL = 219 kips, LL = 49 kips

Base shears are determined in Examples 3-1 and 2.

Due to wind, total base shear on 2 panels is 72 kips with an overturning moment of 1640 kip•ft.

Due to earthquake, total base shear on 2 panels is 301 kips with an overturning moment of 8700 kip•ft.

To find: Allowable bearing pressures p_a and p_a' for a reference footing under the load and soil conditions of footings A1 and A2



Solution:

The allowable bearing pressure p_a is given by Equation 6-15, with $\phi = 0$. Correction factors for shape and depth of founding are applicable for the clay soil; water intrusion is not applicable.

$$p_a = \frac{1}{FS}\left[\gamma D_f N_q' + cN_c'\right]$$

Bearing capacity factors are found in Table 6-1:

$$N_\gamma = 0 \qquad N_q = 1.00 \qquad N_C = 5.142$$

Correction factors for shape are found by interpolation in Table 6-2 for B/L = 14/14 = 1.00:

$$s_q = 1.00 \qquad\qquad s_c = 1.19$$

Correction factors for depth of founding are found in Table 6-3 for $D_f/B = 4/14 = 0.29$:

$$d_q = 1.00 \qquad d_c = 1.12$$

For a commercial office building, an average factor of safety is taken to be 2.5.

The allowable bearing pressure p_a for a reference footing 14 feet square, founded 4 feet below grade is:

$$p_a = \frac{1}{2.5}\left[106 \times 4 \times 1.00 \times 1.00 \times 1.00 + 1250 \times 5.142 \times 1.19 \times 1.12\right]$$
$$= 3600 \text{ psf (for gravity loads)}$$

With wind load added, the allowable bearing pressure p_a' is the same as p_a but with corrections for inclined load. For wind load, the load V to each shear panel is 72/2 = 36 kips. Each footing must be able to sustain this load. The probable gravity load Q on the entire panel at the time of maximum lateral load is:

$$\sum(\text{DL} + 50\%\text{LL}) = (183 + 219) + 0.50(24 + 49) = 439 \text{ kips}$$

$$\text{Hence,} \qquad \frac{V}{Q} = \frac{36}{439} = 0.08$$

Correction factors for inclined load are found in Table 6-5:

$$i_q = 0.90 \qquad i_c = 0.90$$

The allowable bearing pressure p_a' for a reference footing 14 ft. square, founded 4 ft. below grade is computed as:

$$p_a' = \frac{1}{2.5}[106 \times 4 \times 1.00 \times 1.00 \times 1.00 \times 0.90$$
$$+ 1250 \times 5.142 \times 1.19 \times 1.12 \times 0.90]$$
$$= 3240 \text{ psf (for wind load)}$$

With earthquake load added, the allowable bearing pressure p_a' is the same as p_a but with corrections for inclined load. The lateral load V to each shear panel is 301/2 = 151 kips. Each footing must be able to sustain this load. The probable gravity load Q on the entire panel at time of maximum lateral load is:

$$\sum(\text{DL} + 50\%\text{LL}) = (183 + 219) + 0.50(24 + 49) = 439 \text{ kips}$$

$$\text{Hence,} \qquad \frac{V}{Q} = \frac{151}{439} = 0.34$$

(The fact that the ratio V/Q is greater than 0.3 indicates that the footing will slide under this high lateral load. Some reconfiguration of the structure will have to be done in order to reduce the lateral load on this footing. Extending a tie beam to footing A3 is a possible remedy. Whatever remedy is chosen, it is assumed here that the ratio V/Q will be reduced to 0.3 or less.)

Correction factors for inclined load are determined from Table 6-5 for a V/Q ratio of 0.3:

$$i_q = 0.66 \qquad i_c = 0.66$$

The allowable bearing pressure p_a' for a reference footing 14 ft. square, founded 4 ft. below grade is computed as:

$$p_a' = \frac{1}{2.5}[106 \times 4 \times 1.00 \times 1.00 \times 1.00 \times 0.66$$

$$+ 1250 \times 5.142 \times 1.19 \times 1.12 \times 0.66]$$

$$= 2370 \text{ psf (for earthquake load)}$$

The pressures to be carried forward into the footing design are those for a reference footing 14 ft square with 4 ft burial (see Example 9-3).

$$p_a = 3600 \text{ psf}, \qquad p_a' = 3240 \text{ psf for wind,}$$
$$p_a' = 2370 \text{ psf for earthquake}$$

Example 6-10 Continuation of Example 6-9. Allowable bearing pressures p_a and p_a' for interior footings in a braced frame line.

Given : Commercial diaphragm-shearwall concrete office building of Chapter 2, Fig. 2-1.

Loads on interior footing B3 as determined in Examples 2-1, 2-2 and 2-3: DL = 214 kips, LL = 114 kips

A reference footing 10 ft. square is selected, depth of founding 4 ft.

Soil is CH, c = 1250 psf, ϕ is zero. Unit weight γ is 106 pcf. Occasional spring flooding occurs.

To find: Allowable bearing pressure p_a and p_a' for the reference footing under the load and soil conditions of footing B3

Solution:

The allowable bearing pressure pa is given by Equation 6-15, with $\phi = 0$. Correction factors for shape and depth of founding are applicable for the clay soil; water intrusion is not applicable.

$$p_a = \frac{1}{FS}\left[\gamma D_f N_q' + cN_c'\right]$$

Bearing capacity factors are found in Table 6-1:

$$N_\gamma = 0 \qquad N_q = 1.00 \qquad N_c = 5.142$$

Correction factors for shape are found in Table 6-2 for B/L = 10/10 = 1.00

$$s_q = 1.00 \qquad s_c = 1.19$$

Correction factors for depth of founding are found in Table 6-3 for $D_f/B = 4/10 = 0.40$:

$$d_q = 1.00 \qquad d_c = 1.16$$

An average factor of safety for a commercial office building is taken to be 2.5.

The allowable bearing pressure p_a for a reference footing 10 ft. square, founded 4 ft. below grade is:

$$p_a = \frac{1}{2.5}\big[106 \times 4 \times 1.00 \times 1.00 \times 1.00 + 1250 \times 5.142 \times 1.19 \times 1.16\big]$$

$$= 3700 \text{ psf}$$

Interior footings in a diaphragm-shearwall structure are not subject to lateral loads. The allowable bearing pressure p_a' is therefore not applicable to footing B3 (nor to the reference footing).

The pressures to be carried forward into the footing design are those for a reference footing 10 ft square with depth of founding of 4 ft (see example 9-4).

p_a = 3700 psf, p_a' is not applicable

Review Questions

6.1 Throughout all of soil mechanics, it is generally preferred to deal with general comparisons of two or more results rather than deal with the actual absolute values of these same results. Why?

6.2 What general level of accuracy should be expected in calculations involving soils?

6.3 What is the failure mode in a soil when it is loaded by a discrete bearing load such as a footing load?

6.4 Measured from a horizontal surface, what is the angle of shearing failure in a sand?

6.5 Measured from a horizontal surface, what is the angle of shearing failure in a clay?

6.6 State the formula for computing the shear stress at failure in a soil mass under a discrete bearing load. State the formula in terms of the bearing capacity factors.

6.7 List the bearing capacity factors and give the mathematical formulas for calculating them.

6.8 Which term (or terms) of the formula of question 6.6 indicates the effects of footing size?

6.9 Which term (or terms) of the formula of question 6.6 indicates the effects of the unit weight of the soil?

6.10 Which term (or terms) of the formula of question 6.6 indicates the effects of depth of founding in the soil mass?

6.11 Which term (or terms) of the formula of question 6.6 indicates the effects of the cohesion of the soil?

6.12 How are the effects of the shape of the footing incorporated into the allowable bearing capacity of the soil? Depth of founding? Submersion? Lateral loading?

6.13 How is the factor of safety selected for the foundations of a structure?

6.14 Name three factors that will have a significant influence on the selection of a factor of safety for the foundations of a structure.

6.15 In soils classified as clays, the friction component (if any exists) is usually ignored. Why?

6.16 In soils classified as sands, the cohesion component (if any exists) is usually ignored. Why?

6.17 In a sand, if two footings having different loads are designed at the maximum allowable strength of the sand, how will the resulting sizes of the two footings compare, in terms of the two allowable loads? In a clay?

6.18 How is it that every footing size in a group of footings can have a different allowable bearing pressure?

6.19 What means is used to compute individual footing sizes within a group of footings without using a trial-and-correction procedure?

6.20 What effect does the cohesion component of a soil have on the failure angle in shear under a bearing load?

6.21 What is the effect of short-term submergence on the strength of a sand? Of a clay?

6.22 What is the effect of depth of founding on the strength of a sand? Of a clay?

6.23 What is the effect of footing size on the strength of a sand? Of a clay?

6.24 At an extreme, approximately how much of the available strength of a sand will be reduced due to the existence of a lateral load on the sand if cohesion is ignored? Of a clay, if friction is ignored?

OUTSIDE PROBLEMS

6-1 A sand weighing 126 pcf has an angle of internal friction ϕ of 31°. Using Equation 6-4b and assuming that cohesion c is negligible, determine the lateral pressure at a depth of 8 feet as failure impends in this sand.

6-2 A sand weighing 118 pcf has an angle of internal friction ϕ of 28°. Using Equation 6-4b and assuming that cohesion c is negligible, determine the equivalent fluid weight of the sand as failure impends. Compare this result against the results found from the theory of elasticity when failure did not impend.

6-3 A clay weighing 102 pcf has an unconfined compressive strength of 1860 psf with a negligible friction component. Determine the lateral pressure at a depth of 6 feet as failure impends in this clay.

6-4 A stiff clay weighing 108 pcf has a negligible friction component. Using Equation 6-4b, determine the approximate equivalent fluid weight of the clay as failure impends. Compare this result against the results found earlier from the theory of elasticity when failure did not impend.

6-5 A soil has an angle of internal friction ϕ of 28°. Determine the values of the bearing capacity factors and compare your results with the values given in Table 6.1.

6-6 Determine the values of the bearing capacity factors for a pure clay. Compare your results with the values given in Table 6.1.

6-7 A soil weighing 109 pcf has an angle of internal friction ϕ of 24° and an unconfined compressive strength of 480 psf. Determine the contact pressure at failure in this soil when loaded by a strip footing 6 feet wide founded 2 feet deep.

6-8 A GM soil weighing 116 pcf has an angle of internal friction of 34° and negligible cohesion. Determine the bearing capacity of this soil at failure under a strip footing 5 feet wide with $D_f/B = 0.4$.

6-9 A CH soil weighing 108 pcf has an unconfined compressive strength q_u of 2120 psf. A reference footing 8 feet square with a depth of founding of 4 feet is selected for comparison. For this reference footing, determine the allowable bearing pressure p_a with a factor of safety of 3 to failure in shear.

6-10 A stratum of SM soil weighing 114 pcf has a field SPT blow count of 12 at a depth of 9 feet. A reference footing 10 feet square, founded 4 feet deep, is selected for comparison. The site is subject to spring flooding. Determine the allowable bearing pressure p_a for the reference footing, using a factor of safety of 2.5 to failure in shear.

6-11 A stratum of CL soil weighing 102 pcf has a field SPT blow count of 6 at a depth of 8 feet. The site is subject to spring flooding. Lateral *g*-load may be as much as 15% of relevant vertical load. Using a factor of safety of 3 to failure in shear, determine the allowable bearing pressures p_a and p_a' for a reference footing 7 feet square, founded 3 feet deep.

6-12 A stratum of coarse sand weighing 121 pcf has a Dutch cone resistance of 200 kips/ft^2 at a depth of 16 ft. A reference strip footing 5 feet wide founded 2 feet deep is selected for comparison. The site is subject to occasional water table intrusion. Lateral *g*-load may be as much as 13% of relevant vertical load. Using a factor of safety of 2.5 to failure in shear, determine the allowable bearing pressures p_a and p_a' for the selected reference footing.

6-13 A stratum of CH soil weighing 101 pcf has a cohesion of 1496 psf. The site is subject to spring flooding. Lateral g-load may be as much as 9% of relevant vertical load. A factor of safety of 3 to failure in shear has been adopted.

1) Select a size for a reference square spread footing to be founded at a depth of 7 feet in the stratum.
2) Determine the allowable bearing pressures p_a and p_a' for the selected reference square spread footing.
3) Select the size of an actual square spread footing on this soil that will carry a dead load of 176 kips and a live load of 166 kips. (No vertical loads are induced by the lateral loads.)

6-14 A stratum of fine sand weighing 115 pcf has a field SPT blow count of 12 at a depth of 10 feet. The site is not subject to water intrusion. Lateral g-loads may be as high as 10% of relevant vertical load. Factor of safety to shear failure is set at 2.5 for this project.

1) Select a size for a reference square spread footing to be founded 4 feet deep.
2) Determine allowable bearing pressures p_a and p_a' for the selected reference spread footing.
3) Select a suitable size for an actual square spread footing on this soil that will carry a dead load of 196 kips and a live load of 171 kips. (No vertical loads are induced by the lateral loads.)

6-15 A stratum of clay weighing 104 pcf has an unconfined compressive strength of 2420 psf. The site is subject to spring flooding. Lateral *g*-load may be as much as 20% of relevant vertical load. A factor of safety of 3 has been set for the foundations of this project.

1) Select a size for a reference strip footing to be founded 2.5 feet deep.
2) Determine the allowable bearing pressures p_a and p_a' for the reference strip footing.
3) Select a suitable width for an actual strip footing that will carry a dead load of 7 kips/ft and a live load of 4 kips/ft. (No vertical loads are induced by the lateral loads.)

6-16 A stratum of sand on a building site weighs 124 pcf and has an angle of internal friction of 32°. The site is subject to spring flooding. Lateral *g*-load may be as much as 18% of relevant vertical load. A factor of safety of 2 has been set for the foundations of the project.

1) Select a size for a reference strip footing to be founded 2.5 feet deep.
2) Determine the allowable bearing pressures p_a and p_a' for the reference strip footing.
3) Select a suitable width for an actual strip footing that will carry a dead load of 7 kips/ft and a live load of 4 kips/ft. (No vertical loads are induced by the lateral loads.)

Chapter 7

SETTLEMENT OF FOUNDATIONS ON A SOIL MASS*

Consolidation and Settlement in Clays

Soils are not elastic. Although there is some rebound in a soil when a compressive load is removed, the rebound is quite small; most of the compressive deformation is permanent. To avoid any inference that deformations are elastic, all such deformations in soils are termed herein as *settlements*.

A typical clay sample is shown in Fig. 7-1, magnified many times. The clay particles are shown as thin flat particles (like leaves) touching each other in random order. As seen in an electron microscope, however, the appearance of a clay particle is more like an old battered book with pages sticking out raggedly in all directions. The tiny interstices between particles contain trapped water and occasional pockets of trapped air.

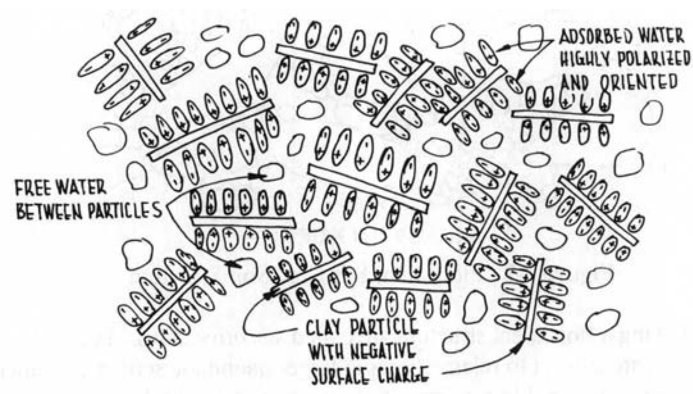


Figure 7-1 Adsorbed Water on Clay Particles

* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

Clay particles themselves are somewhat tough and flexible. Unlike sand grains, they can be deformed or bent before they break. As they bend, however, they develop more and more resistance to further deformation.

The interstices, or pores, surrounding the clay particles are extremely small. The porewater is in thin layers and strips and is partially bonded to the clay. The partially bonded porewater closest to the clay particles is quite viscous, acting like a very "thick" fluid, becoming "thinner", or less viscous, as distance from the particle increases. The water is trapped, of course, but the thinner water can be forced out of the clay by the introduction of a pressure gradient across the deposit.

Depending on the way a clay is deposited, the clay particles may settle into a very loose *flocculent* or *honeycomb* structure as shown in Fig. 7-2[36]. In slowly deposited sediments, the buildup of the clay deposit can occur with very high void ratios as shown. When loaded, the clay structure can collapse into the large void spaces, producing unusually large and rapid settlements.

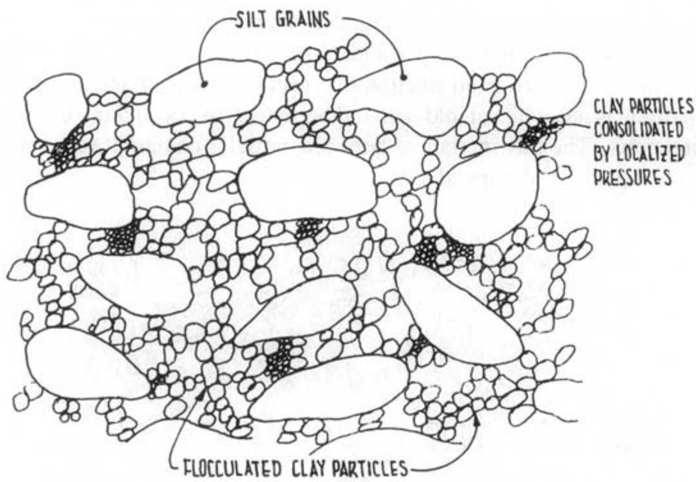


Figure 7-2 Flocculent Silt-Clay Soil Structure

Clay soils having a flocculent structure are called *sensitive* clays. Typically, sensitive clays are subject to relatively rapid large-magnitude settlements under load. Mechanically remolding a deposit of sensitive clay will destroy the flocculent structure, restoring the clay to a more dense state. Such remolding, however, will produce a large reduction in volume and a reduction in strength.

Almost all clay soils exhibit some degree of sensitivity, which means that as a rule, almost any clay will lose some degree of strength when remolded. The effects of a low to moderate degree of sensitivity are usually of little consequence, however, and are usually ignored. The only soils considered in this elementary text are these clays of low-to-moderate sensitivity.

While not rare, clays having high levels of sensitivity are only occasionally encountered in routine practice. When encountered, however, they should be regarded with caution; these highly sensitive clays can be adversely affected by external influences at a particular site. In some cases, for example, driving piles through a deposit of such clay can cause a localized remolding within the deposit, producing unpredictable settlement patterns. Also, the effects of seismic shaking can produce remolding and rapid settlement of foundations in such clays. Specialized textbooks in soil mechanics treat the subject of foundations on sensitive clays in considerable detail[12,14].

A clay that has never been subjected to any pressure greater than the pressure it now sustains, whether from overburden or from foundation loads, is said to be *normally consolidated*. Such lightly condensed soils, compressed only by their own weight, occur in delta regions of large rivers when clay deposits are laid down by periodic flooding. The response of these relatively soft compressible soils to a foundation load is reasonably predictable in comparison to other types of clay.

A normally consolidated clay can often be identified by its in-place moisture content. If the in-place moisture content is just below the liquid limit, the clay is very likely a normally consolidated clay.

Degree of Consolidation

A load placed on any clay soil, including a normally consolidated clay, will at first be carried entirely by an increase in pressure in the trapped porewater, as shown in the time-pressure curve of Fig. 7-3. Over the following months or years, the "thinner" less viscous water is extruded away from the region of increased pressure. As that happens, the load is slowly transferred to the "thicker" water

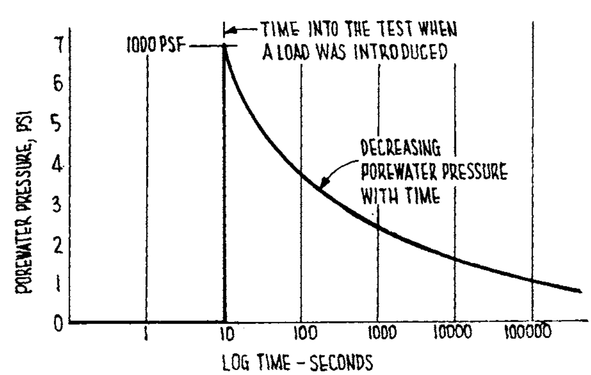


Figure 7-3 Time-Pressure Curve for Load on a Clay Soil

and to the clay particles until equilibrium is reached under the increased pressures and higher viscosities.

When equilibrium is reached, the clay is said to be *100% consolidated at the increased pressure*. At 100% consolidation, the increase in porewater pressure has been completely dissipated; the porewater pressure has returned to its original state. The *degree of consolidation U%* at any time is the percent complete at that time.

The process of making the very slow transfer of load from the porewater to the clay particles, with its corresponding decrease in volume, is given the generic name *consolidation*. It is the mechanism whereby clay deforms under load. It explains why deformations occur so slowly in clays, sometimes taking several months or even years before settlements finally stabilize under an increase in load.

Consolidation settlements in clays are viewed as a long-term phenomenon. Under short-term loads such as wind or earthquake, there simply is not enough time for the porewater to escape outward through the tiny pore spaces. As indicated in Fig. 7-3, all of the increase in pressure from short-term loads can be considered to be sustained entirely by the porewater.

Too, since the voids in clay soils are usually filled with water (or nearly so), and since water is relatively incompressible compared to soil, very little deformation will accompany short-term loads on clay soils. Where air pockets exist, the entrapped air will of course be compressed by the increased pressure, resulting in a small amount of short-term deformation. When the load is released, the air immediately expands, thus producing a small amount of "elastic" rebound. Otherwise, the trapped porewater will sustain the entire short-term load with very little deformation.

Some examples will illustrate some uses of porewater pressure in estimating such things as degree of consolidation and settlements.

Example 7-1 Porewater pressure, settlements and degree of consolidation.

Given : Footing on a stratum of clay soil with a pressure gage installed immediately below the footing to measure porewater pressure. A load is placed on the footing, which causes an increase in porewater pressure of 6.2 psi. At the end of 6 months, the increase in pressure has dropped to 0.96 psi.

To find: The degree of consolidation at 6 months.

Solution:

At the end of 6 months, the increase in pressure has dropped by an amount (6.2 - 0.96)/6.2 or an 84.5% drop, that is, 84.5% of the porewater that is going to be extruded out of the soil has been extruded out. The degree of consolidation $U\%$ is therefore 84.5% at the end of 6 months.

Example 7-2 Porewater pressure, settlements and degree of consolidation

Given : Footing on a stratum of clay soil with a pressure gage installed immediately below the footing to measure porewater pressure. A load is placed on the footing, which causes an increase in porewater pressure of 5.8 psi. At the end of 3 months, the increase in porewater pressure has dropped to 4.3 psi.

The settlement of the footing at that time is measured at 0.45 in.

To find: The total settlement to be expected

Solution:

At the end of 3 months, the increase in pressure has dropped by an amount (5.8 - 4.3)/5.8 or a 25.9% drop, that is, 25.9% of the porewater that is going to be extruded out has been extruded out. As a corollary, 25.9% of the change in volume (and therefore the settlement S) that is going to occur has occurred by this time, or

$$0.259 \times S = 0.45 \text{ in.}$$
$$S = 1.74 \text{ in.}$$

A total settlement of 1.74 in. can be expected at some point in the future.

Example 7-3 Porewater pressure, settlement and degree of consolidation

Given : Spread footing 6 ft. square on a stratum of clay soil. The footing is loaded relatively quickly by a load of 70 kips. Settlement S is monitored monthly thereafter and found to be:

At 1 month : S = 2.30 in.
At 2 months : S = 3.45 in.
At 3 months : S = 4.31 in.
At 4 months : S = 4.60 in.
At 5 months : S = 4.60 in.

To find: Porewater pressure at the beginning of the test and at the end of each month thereafter.

Solution:

At the start of the test, the porewater carries the entire load of the footing. The increase in porewater pressure Δp is computed as:

$$\Delta p = \frac{P}{A} = \frac{70{,}000}{6 \times 6} = 1944 \text{ psf}$$

$$\Delta p = 13.5 \text{ psi at the start of the test}$$

At one month, the settlement is 2.3/4.6 = 0.50 or 50% complete. The increase in porewater pressure has similarly dropped to:

At 1 month :	Δp	=	[(4.60 - 2.30)/4.60]13.5	=	6.75 psi
At 2 months:	Δp	=	[(4.60 - 3.45)/4.60]13.5	=	3.38 psi
At 3 months:	Δp	=	[(4.60 - 4.31)/4.60]13.5	=	0.85 psi
At 4 months:	Δp	=	[(4.60 - 4.60)/4.60]13.5	=	0

Overconsolidated Clay

It is again noted that there is a distinct reduction in the volume of the clay as it consolidates. The loss in volume is of course the volume of porewater that has been extruded outward and lost, with a corresponding reduction in the volume of voids. Such a flow of porewater in a stratum of clay is shown in Fig. 7-4. The clay particles themselves, along with the remaining "thicker" water, are in a state of increased stress with a reduced volume.

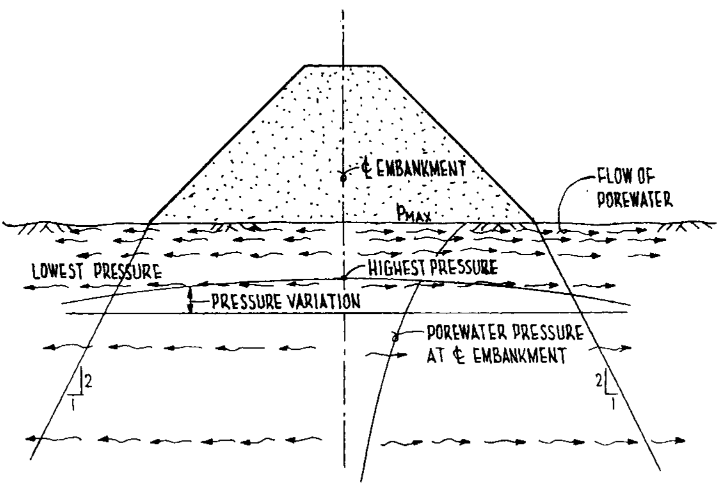


Figure 7-4 Flow of Porewater due to Increased Pressure

When a clay stratum has been loaded by thousands of feet of overburden (or ice-age glaciers) and when this load is eventually eroded (or melted) away, the release of the load creates restoring forces in the clay that can be quite large. The clay remains compressed, like a compressed spring, unable to expand until conditions permit an ingress and restoration of some part of the porewater that had been extruded eons ago.

In this precompressed state, the clay is said to be *overconsolidated.* An overconsolidated clay is defined as any clay that has been subjected to a greater level of consolidation pressure in its past than it now experiences. Of the two possible cases of consolidation that can be found in clay soils, either normal consolidation or overconsolidation, the more common case by far is that of overconsolidation.

An overconsolidated clay can often be identified by its in-place moisture content. If the in-place moisture content is significantly below the liquid limit, the clay is very likely an overconsolidated clay. In some cases, the in-place water content of an overconsolidated clay can even be below the plastic limit.

When a source of water is eventually made available, the ingress of water allows the particles to unbend and to return closer to their original configuration. Such expansion typically happens in small pockets at random locations, producing unpredictable localized heaving. Such small isolated pockets of expanding soil can devastate buildings built on the stratum.

In some geologically overconsolidated clays, the expansion and heaving of the clay can be a serious and ever-present problem. A well-known example of such a clay is the Red Permian clay of Texas, Oklahoma and Kansas. Horror stories about heave in the Red Permian clay are well known to the local foundation designers.

A clay can also become overconsolidated by severe drying, or *desiccation.* As the clay dries, the capillary tension in the porewater can become quite large and can cause severe amounts of volume change due to shrinkage. Desiccation is one of the more common causes of overconsolidation.

Since the change in volume due to desiccation occurs in three directions, a desiccated clay will shrink laterally, which will in turn produce vertical cracking. As indicated in Fig. 7-5, cracks may be several inches wide, up to several feet deep and several yards long, forming permanent planes of weakness in the clay mass.

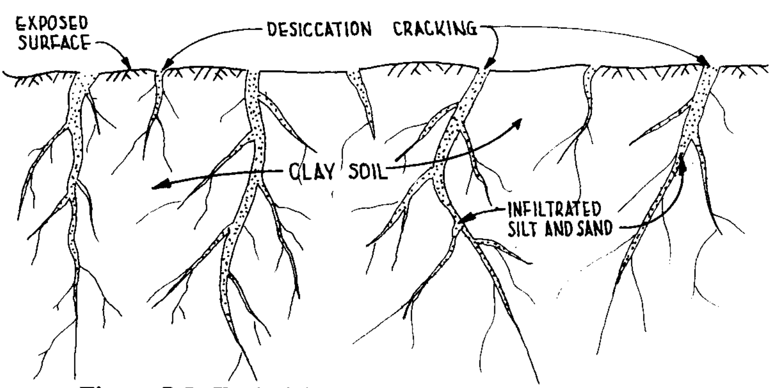


Figure 7-5 Typical Crack Penetration in Desiccated Clay

Desiccated clays are known for their rapid and erratic expansion when a water source finally becomes available. The criss-crossing pattern of open cracks will commonly become filled with windblown silt. Since the silt is more permeable than clay, the filled cracks thus provide a permanent means of ingress for water. Such an ingress of water can of course produce rapid and unpredictable large-scale heaving; the effects on structures can be crippling.

The Consolidation Test for Clay Soils

As with other engineering materials such as steel or concrete, the calculation of deformations in soils begins with a stress-strain curve. In soils, however, the stress-strain curve takes a somewhat different form due simply to the nature of soil. The strain is best viewed as a change in void ratio and the stress is best viewed as the log of the vertical pressure p_1. The reason for using a log scale for pressures comes as a result of the extremely slow rate of consolidation as porewater pressure approaches zero.

Some typical stress-strain curves for plastic soils are shown in Fig. 7-6. Such curves are usually called *e*-log *p* curves rather than stress-strain curves. As indicated, the curves are those for a normally consolidated clay, an overconsolidated clay and a sensitive clay. Shown in dashed lines is an idealized curve for the sensitive clay after it has been remolded.

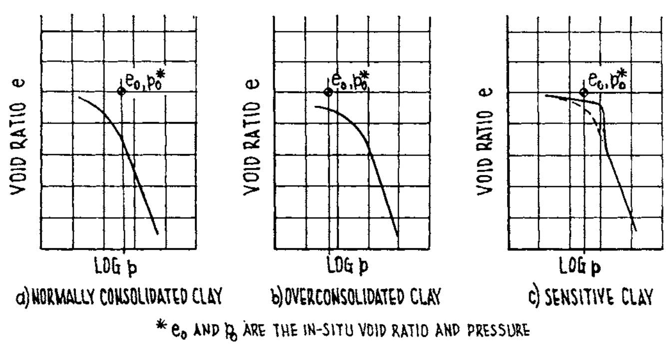


Figure 7-6 Typical *e*-log *p* Curves

Note that the *e*-log *p* curve of the sensitive clay in Fig. 7-6 has a very steep slope when the flocculent structure first begins to break down. The effect is somewhat akin to the "plastic range" in steel, where deformations increase with little or no increase in load. The settlement of footings on such sensitive clays can be quite large and can occur quite rapidly.

All deposits of clay begin their geologic lives as normally consolidated clays. It should not be inferred, however, that pressures in a deposit of normally consolidated clay are low. While it is true that pressures have never been higher than they are now, the pressures at the bottom of a deep deposit of normally consolidated clay can be quite high.

In a consolidation test, a sample of the clay soil is fitted snugly into a circular confining ring and is loaded by a uniform pressure as indicated schematically in Fig. 7-7. The apparatus allows free flow of water out of the sample at top and bottom.

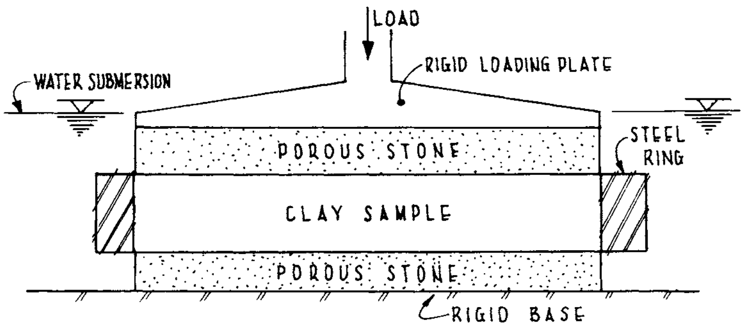


Figure 7-7 Schematic Diagram of a Consolidation Test

Before the test is started, the maximum anticipated increase in the soil pressure is established. Also, the *in situ* void ratio e_0 and the *in situ* overburden pressure p_0 are determined. At the start, the sample is loaded until the *in situ* pressure p_0 on the sample is restored; deformations are allowed to come to rest. The void ratio at that point is taken to be e_0.

The actual test procedure then consists of a step-by-step incremental increase in load on the confined sample until the anticipated maximum pressure on the soil has been reached. At each step of loading, the compressive deformations at various times are recorded for that increment of load. Deformations are allowed to come to a stop before the next increment is added. The procedure is repeated for a number of increments of load until the anticipated maximum pressure is reached.

As an optional part of the test, the rebound deformations of the clay may also be included. At some point well into the procedure, preferably about halfway, all of the added increments of load are removed. The sample will then undergo "rebound" back to pressure p_0. When deformations stabilize, the vertical deformation at this rebound position is recorded. All of the increments are then replaced. When deformations again stabilize, the deformation is recorded and the regular test procedure is resumed. These results will be used later to reproduce the rebound-reload curve of the soil. (Technically speaking, the sample is now overconsolidated.) When the test is complete, results at each increment of load are plotted, providing a graph similar to that of Fig. 7-8a.

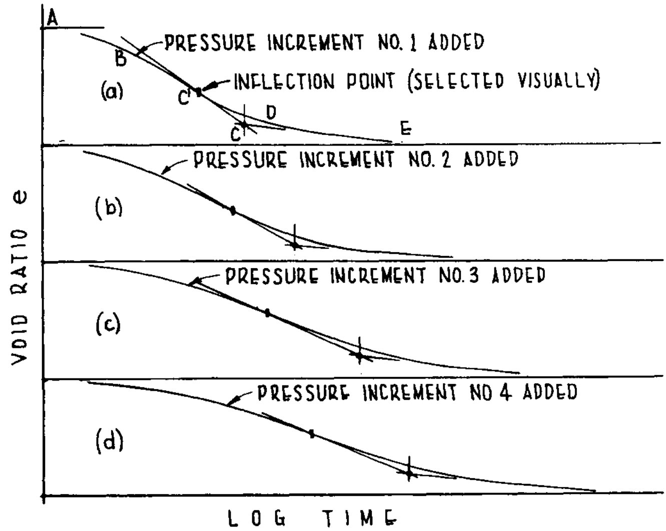


Figure 7-8 *e*-log time Consolidation Curves

The set of curves shown in Fig. 7-8 are commonly called the *e*-log time curves. A complete set of these *e*-log time curves is necessary, one curve for each increment of pressure up to the maximum anticipated pressure.

There are two types of consolidation reflected in the *e*-log time curves of Fig. 7-8a. The first type is the primary consolidation, defined by the initial portion of the curve having a steep downward slope, ABC'. The second type is the secondary consolidation, defined by the final portion of the curve having a shallow upward asymptotic curve C'DE.

A graphical method is used to find the theoretical point C' where the two curves meet. As shown in Fig. 7-8a, a line is drawn through the inflection point C' tangent to both the upper part of the curve and the lower part of the curve. Another line is drawn tangent to the secondary consolidation curve along its straightest portion. The intersection of these two lines occurs at point C, which is taken to be the point where primary consolidation is 100% complete and secondary consolidation begins.

Secondary consolidation is of interest only rarely. The time involved can be upward of 50 to 100 years, far beyond the service life of today's commercial structures. In general, references to consolidation will mean only primary consolidation, or that part of the curve shown as ABC in Fig. 7-8a.

It is emphasized that Fig. 7-8a is drawn using a beginning pressure p_0, with an added increment of pressure producing an increase to $p_0 + \Delta p$. At that pressure, the clay reaches 100% consolidation when void ratio and pressure reach point C. At this point, essentially all the water has been forced out of the clay that is going to be forced out at this increase in pressure.

The initial portion of the primary consolidation curve (up to 50% to 60% of the total) can be considered to be essentially a second order parabola. For all increments of load after the first one, the void ratio at the initial point A corresponding to 0% consolidation can be projected as indicated in Fig. 7-9.

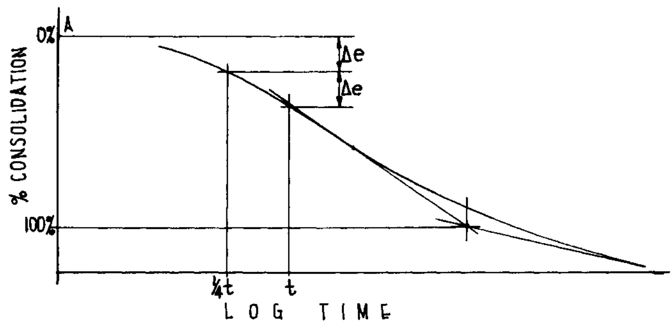


Figure 7-9 Projected Point of Zero Consolidation

To project the curve of Fig. 7-9 to point A, find values of e and t for some arbitrary point in the first half of the curve. Find the point $^1/_4t$ as indicated and draw a horizontal line through that point. Lay off the distance Δe between t and $^1/_4t$ as indicated. Then draw a second horizontal line at a distance Δe above the first horizontal line. The intersection of this second horizontal line with the vertical ordinate is the point of zero consolidation.

Once the points of 0% consolidation and 100% consolidation are known, the scale for the vertical ordinate is readily set. With a 0% to 100% scale on the vertical ordinate, each e-log time curve represents a consolidation-time curve at the given incremental increase in pressure. This concept is developed further in the next section.

Once the set of e-log time curves are complete, the final consolidation curve can be drawn. The values of e and p corresponding to 100% consolidation are taken from the e-log$p$ curves and plotted as a separate graph. The end result of the consolidation test is the consolidation curve shown in Fig. 7-10 above.

The consolidation curve is commonly called the e-log$p$ curve. The e-log$p$ curve for clay is roughly equivalent to the stress-strain curve for steel or timber. Each point of the curve defines the void ratio e at 100% consolidation under the indicated vertical pressure. It should be noted, however, that a consolidation curve is independent of time; the time required to reach 25%, 50% or even 100% consolidation under various pressures is not indicated.

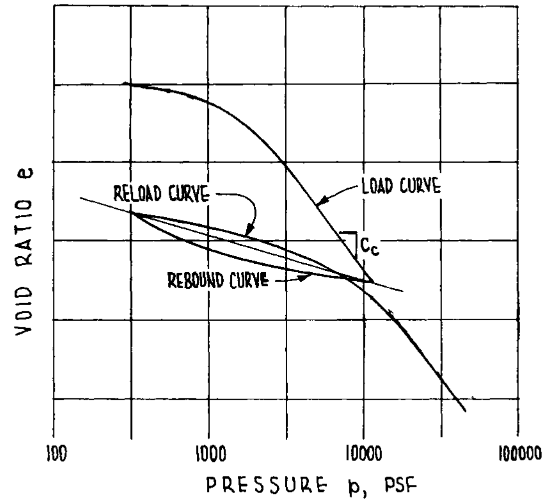
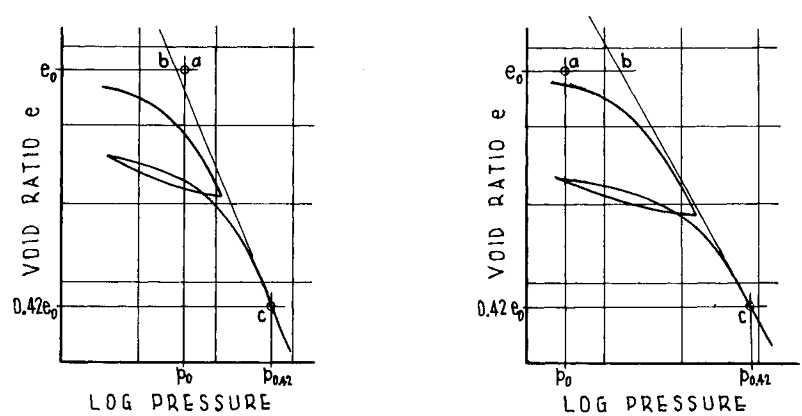


Figure 7-10 Typical Consolidation Curve

As a side benefit, the consolidation curve can be used to determine whether the clay it represents is normally consolidated or overconsolidated. Two typical laboratory consolidation curves are shown in Fig.7-11. One curve is a normally consolidated clay and one is an overconsolidated clay. A simple observation will reveal which is which.



a) Normally consolidated clay b) Overconsolidated clay

Figure 7-11 e-log$p$ Curves for Consolidated Clays

At the higher pressures of both curves in Fig. 7-11, there is a portion of the curve that is essentially a straight line. That straight-line portion of the curve is extended downward to $0.42e_0$ (point c) and upward to e_0 (point b) The point p_0,e_0 is then plotted on the graph, shown as point a in both curves of Fig.7-11. If the point a lies to the right of the sloping straight line, the clay is normally consolidated; if it lies to the left, the clay is overconsolidated.

The rebound-reload curve may or may not have been included in the original consolidation test. The foregoing procedure for identifying the type of consolidation can be used regardless whether or not the rebound-reload curve is included.

Comparative Time-consolidation Relationships

It was observed earlier that the consolidation curve has been made independent of time. The void ratio shown in the consolidation curve is the void ratio taken from the *e*-log time curves when the consolidation was 100% complete at the indicated pressure. However, the consolidation curve by itself offers no indication of the time required for the sample to reach 100% consolidation.

It can sometimes happen that settlements must be determined for a particular increase in load Δp at some point in time other than at 100% consolidation. In such cases, a separate consolidation test can be run at the anticipated pressure increment. The e-log time curve so generated is converted to a consolidation-time curve as described in the last section. For this curve, as shown in Fig. 7-12, the ordinate is the degree of consolidation (in %) rather than void ratio.

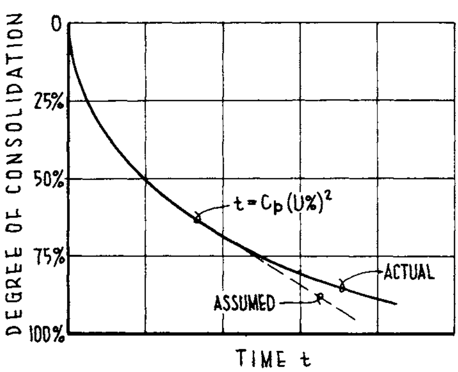


Figure 7-12 Consolidation-time Curve

It was noted in the last section that the *e*-log time curve (now the consolidation-time curve) can be closely approximated as a second degree parabola. The curve of Fig. 7-12 can therefore be expressed as

$$t = C_P\left(U\%\right)^2 \tag{7-1}$$

where C_P is the constant of proportionality, a physical property of each soil, and $U\%$ is the degree of consolidation at time t.

By itself, Equation (7-1) has little use, since the constant C_P is unknown and is different for every soil. However, the degree of consolidation $U_1\%$ and $U_2\%$ at two times t_1 and t_2 can be stated as:

$$t_1 = C_P\left(U_1\%\right)^2 \quad \text{and} \quad t_2 = C_P\left(U_2\%\right)^2$$

Dividing the first equation by the second eliminates the unknown constant C_P and permits a comparison to be made of the two values of $U\%$:

$$\frac{t_1}{t_2} = \frac{\left(U_1\%\right)^2}{\left(U_2\%\right)^2} \tag{7-2}$$

Given any three of the four variables in Equation (7-2), the fourth may be readily determined. An example will illustrate one use of such a relationship.

Example 7-4 Use of parabolic ratios

Given : A soil sample in a consolidation test reaches 100% consolidation in 206 seconds under an increase in pressure of 1200 psf.

To find: The time when the sample reached 30% consolidation

Solution:

The time is found by parabolic ratios:

$$\frac{t_1}{t_2} = \frac{\left(U_1\%\right)^2}{\left(U_2\%\right)^2}, \qquad \frac{206}{t_2} = \frac{100^2}{30^2}, \qquad t_2 = 18.5 \text{ secs}$$

The sample reached 30% consolidation at 18.5 seconds into the test

A second set of observations will serve to expand the applicability of Equation (7-2). It is observed that settlement is directly proportional to consolidation. That is, when consolidation is 20% complete, 20% of the porewater that is going to be extruded out has been extruded out. Correspondingly, 20% of the reduction in volume has occurred, therefore 20% of the settlement has occurred, and the initial increase in porewater pressure has dropped 20%. At any given increase in pressure, Δp, Equation (7-2) may therefore be expressed either in terms of U% or of settlement S:

$$\frac{t_1}{t_2} = \frac{(U_1\%)^2}{(U_2\%)^2} = \frac{S_1^2}{S_2^2} \tag{7-3}$$

A further observation will permit Equation (7-3) to be used to compare settlements recorded in a laboratory sample to the settlements expected in the field *at the same increase in pressure.* It has been observed (and verified) that the time required for the porewater to be extruded out of a layer or stratum of water is proportional to the square of the distance the porewater must travel to a free surface[18]. This concept of the "travel distance" is shown as distance H in Fig. 7-13.

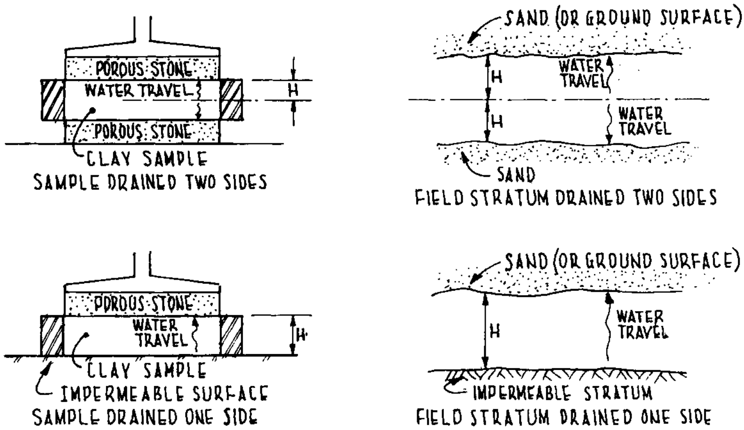


Figure 7-13 Travel Distances for Consolidation

It is concluded that under a given increase in pressure Δp, the relationship given by Equation (7-2) may be expressed either in terms of consolidation $U\%$ or settlement S or the travel distance H, that is,

$$\frac{t_1}{t_2} = \frac{(U_1\%)^2}{(U_2\%)^2} = \frac{S_1^2}{S_2^2} = \frac{H_1^2}{H_2^2} \tag{7-4}$$

In the sample of Fig. 7-13, the farthest distance the porewater must travel to a free surface is the distance H, as shown. It is this value of travel distance H that is used in Equation (7-4).

The reason that Equation (7-4) is written as a rather strange succession of equalities is to emphasize that each of the variables t, $U\%$, S and H are interrelated; all four quantities are those corresponding to a single increment of pressure Δp. It also emphasizes that variations with time t are parabolic, but that variations between $U\%$, S and H are linear.

The fact that all the variables in Equation (7-4) are those at a particular value of Δp becomes an important limitation in the use of the equation. The use of Equation (7-4) is limited to comparing time-consolidation relationships only where the pressure increase Δp is essentially constant for every particle of soil throughout the thickness of the stratum being considered.

Where pressures vary with depth, as in a Boussinesq pressure bulb, Equation (7-4) may still be used, but only to compare time-consolidation relations involving particles at the same depth in the pressure bulb. It may *not* be used to compare time-consolidation relations between particles at one depth to those at another depth, since the two pressure increments Δp will not be the same.

It should be apparent from an examination of Equation (7-4) that the longest settlement times at any depth below the contact surface of a footing will be those where pressures are highest. Very simply, more water will be extruded at the higher pressures, creating larger vertical strains, thus creating higher settlements and in turn requiring more time. The point of greatest interest, therefore, would be that just below the contact surface, where pressures and consolidation times are highest.

Consequently, if one were to determine the consolidation time just below the contact surface, the result would be the longest consolidation time anywhere in the bulb, which is generally the time of greatest interest. To apply such an idea, however, would require that a time-consolidation test be run for each footing, a requirement that renders the idea impractical.

The comparison of time and travel distance between the lab sample and the field stratum under the same increase in pressure Δp is then:

$$\frac{t_{LAB}}{t_{FIELD}} = \frac{H_{LAB}^2}{H_{FIELD}^2} \tag{7-5}$$

where t_{LAB} = time required to achieve a given degree of consolidation U_l at a given increase in pressure Δp_l in the lab sample

t_{FIELD} = time required to reach the same degree of consolidation U_l at the same increase in pressure Δp_l in the field stratum

H_{LAB} = farthest distance a particle of porewater must travel to a free surface in the lab sample

H_{FIELD} = farthest distance a particle of water must travel to a free surface in the field stratum

Some examples will illustrate the use of Equation (7-4)

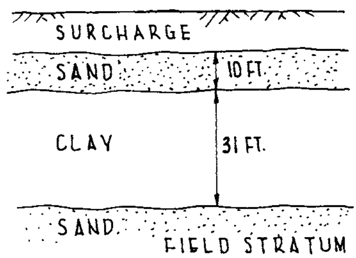
Example 7-5 Comparisons of time and consolidation

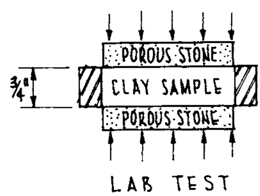
Given : A stratum of clay 31 ft thick is overlain and underlain by deposits of sand as shown in the sketch. The *in situ* pressure toward middepth of the clay stratum is about 3000 psf or so. A surcharge is to be placed over the area, increasing the pressure on every particle in the stratum of clay by an increment of 1000 psf.

A sample of the clay 0.75 in. thick is tested in a consolidation test, drained both sides.

In going from 3000 psf to 4000 psf, the lab sample reached 50% consolidation in 73 seconds and 100% consolidation in 4.9 minutes. (It should be noted that the result would be the same regardless what starting pressure is used.)

To find: Time for the field stratum to reach 50% consolidation and 100% consolidation under the same increment of increase in pressure.





Solution:

The solution is given by parabolic ratio:

$$\frac{t_{LAB}}{t_{FIELD}} = \frac{H_{LAB}^2}{H_{FIELD}^2}$$

At 50% consolidation,

$$\frac{73}{t_{FIELD}} = \frac{(0.75/2)^2}{(31\times 12/2)^2}$$

$$t_{FIELD} = 17.9\times 10^6 \text{ sec} = 207 \text{ days} = 7 \text{ months}$$

At 100% consolidation,

$$\frac{4.9}{t_{FIELD}} = \frac{(0.75/2)^2}{(31\times 12/2)^2}$$

$$t_{FIELD} = 1.2\times 10^6 \text{ min} = 2.29 \text{ years}$$

Example 7-6 Comparisons of time and consolidation

Given : Conditions of Example 7-5 but for this case the field stratum is underlain by an impermeable shale

To find: Times for the field stratum to reach 50% and 100% consolidation.

Solution:

The only change in the solution is in the distance the porewater must travel in the field stratum. Since the porewater can now travel in only one direction, H_{FIELD} now becomes 31 ft rather than 15.5 ft.

At 50% consolidation,

$$\frac{73}{t_{FIELD}} = \frac{(0.75/2)^2}{(31\times 12)^2}$$

$$t_{FIELD} = 71.8\times 10^6 \text{ sec} = 831 \text{ days} = 27.7 \text{ months}$$

At 100% consolidation,

$$\frac{4.9}{t_{FIELD}} = \frac{(0.75/2)^2}{(31\times 12)^2}$$

$$t_{FIELD} = 4.8\times 10^6 \text{ min} = 9.2 \text{ years}$$

The results of Examples 7-5 and 7-6 indicate that improvements in field drainage may well be worth considering when consolidation times are too long to be practical.

Example 7-7 Settlement time vs. construction time

Given : Stratum of clay 26 ft thick at the surface of a construction site, having a unit weight of 112 pcf, underlain by sand.

A permanent fill of imported sand weighing 115 pcf is to be placed 10 ft thick over the area where a structure is to be built.

Arbitrarily, the consolidation time is calculated at midheight of the clay stratum where the *in-situ* pressure is 112x26/2 = 1460 psf. (The consolidation time depends only on the *increase* in pressure, not the *in-situ* pressure.) The incremental increase in pressure at every point in the stratum due to the added fill is 115x10 = 1150 psf.

Based on the in-place pressure and the added fill weight, a consolidation test, drained two sides, was run on a sample of clay 0.75 in. thick, starting at a pressure of 1460 psf, then increasing it by an increment of 1150 psf to a total pressure of 2610 psf; 50% consolidation was reached in 48 seconds and 100% consolidation was reached in 244 seconds.

To find: a) If construction is delayed until the clay reaches 50% consolidation, how much delay is incurred?

b) If construction time is scheduled for the 12 months following this initial delay, will the underlying clay reach 100% consolidation by the time construction is finished?

Solution:

a) At 50% consolidation, the time is found by parabolic ratio:

$$\frac{t_{LAB}}{t_{FIELD}} = \frac{H_{LAB}^2}{H_{FIELD}^2}, \qquad \frac{48}{t_{FIELD}} = \frac{(0.75/2)^2}{(26 \times 12/2)^2}$$

$$t_{FIELD} = 8.31 \times 10^6 \text{ secs} = 96 \text{ days}$$

The delay until the underlying clay reaches 50% consolidation is 96 days.

b) At 100% consolidation,

$$\frac{t_{LAB}}{t_{FIELD}} = \frac{H_{LAB}^2}{H_{FIELD}^2}, \qquad \frac{244}{t_{FIELD}} = \frac{(0.75/2)^2}{(26 \times 12/2)^2}$$

$$t_{FIELD} = 42.2 \times 10^6 \text{ secs} = 489 \text{ days}$$

Total elapsed construction time: 96 + 365 = 431 days, slightly less than the consolidation time of 489 days.

The use of Equation (7-4) also allows valuable predictions to be made concerning relative settlements. For example, when a building is to be built over a clay deposit, the reduction in void ratio and corresponding settlements due to the building weight can be estimated by some convenient method (such as one of those presented later in this chapter). The building site can then be preloaded with fill material (surcharged) over the several months that the project is in the design and bidding stages. The amount of surcharge is carefully selected such that it will produce the same reduction in void ratio that the building weight would eventually produce. When it is time to start construction, the surcharge is removed and the building is built on the preconsolidated soil; at that time, the anticipated settlements will already be complete.

The construction practice just described is examined further in the following example.

Example 7-8 Settlement time vs. construction time

Given : Building site with a surface stratum of clay soil 18 ft. thick weighing 115 pcf. The stratum is underlain by a dense sand.

The building site is to be surcharged with a sand fill 14 ft. thick, weighing 132 pcf.

Arbitrarily, the incremental increase in pressure is computed at midheight of the clay stratum where the *in-situ* pressure is 115 x 9 = 1035 psf. (The consolidation time depends only on the *increase* in pressure, not the *in-situ* pressure.) The incremental *increase* in pressure at every point in the stratum due to the added fill is 132 x 14 or 1850 psf, for a total of 2885 psf

Based on the *in-situ* pressure and surcharge weight, a consolidation test, drained two sides, was run on a sample of clay 0.75 in. thick, starting at a pressure of 1035 psf and increased by an increment of 1850 psf to a total of 2885 psf. 100% consolidation was reached in 258 seconds.

An estimate of the settlement of the building due to its sustained long-term load was made, indicating that total settlement of 2.99 in. can be expected.

To find: 1) Estimate of the settlement to be expected at the end of 60 days.

2) Estimate of the time required to reach a settlement of 2 in.

3) Estimate of the time required to reach 50% consolidation.

Solution:

The first part of the calculation is to find the time required for the clay to reach 100% consolidation under the given surcharge pressure. Equation (7-4) is used.

$$\frac{t_{LAB}}{t_{FIELD}} = \frac{H_{LAB}^2}{H_{FIELD}^2}, \qquad \frac{258}{t_{FIELD}} = \frac{(0.75/2)^2}{(18\times 12/2)^2}$$

$$t_{FIELD} = 248 \text{ days}$$

It will take 248 days to settle 2.99 in.

a) At the end of 60 days,

$$\frac{t_{60}}{t_{248}} = \frac{S_{60}^2}{S_{248}^2}, \qquad \frac{60}{248} = \frac{S_{60}{}^2}{2.99^2}$$

$$S_{60} = 1.47 \text{ in. settlement at 60 days}$$

b) For a settlement of 2 in.,

$$\frac{t_2}{248} = \frac{2^2}{2.99^2}, \qquad t_2 = 111 \text{ days to settle 2 in.}$$

c) For 50% consolidation,

$$\frac{t_{50}}{248} = \frac{50^2}{100^2}, \qquad t_{50} = 62 \text{ days for 50\% consolidation}$$

In addition to its local effects on foundation settlements, the long-term consolidation of a clay soil under a long building can produce an area-wide *distortion* settlement under the building. A typical example of distortion settlement is shown in Fig. 7-14.

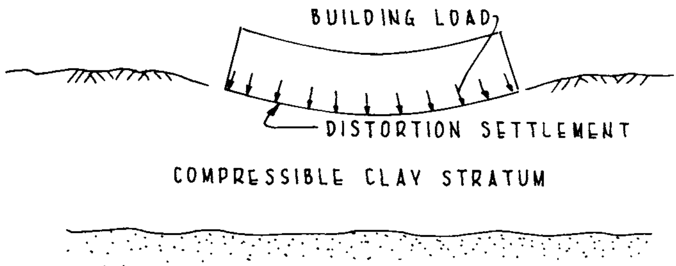


Figure 7-14 Distortion Settlement in Clay

Due to the overlapping pressure bulbs (See Chapter 5, Fig. 5-17), there will be a buildup of pressure under the center of the building. When the consolidation is eventually complete, the reduction in volume (consolidation) at the center of the building will therefore be greater than that at the edges. Due to such differences in settlements, a long building founded on clay will typically cup downward at its center, as shown in Fig. 7-14.

The magnitude of distortion settlements can be severe for large long structures such as dams, embankments, levees and other works. For the usual run of small buildings, however, distortion settlements are rarely large enough to be of concern. Notable exceptions can be found in some of the older "row" buildings on the normally consolidated clays of New Orleans, where distortion settlements are readily visible.

Fragmentation and Settlement in Sands

A typical sample of sand is shown schematically in Fig. 7-15 at a greatly enlarged scale. The grains of sand are shown as irregular but generally rounded particles, contacting each other at a single point. The void spaces may or may not be filled with water.

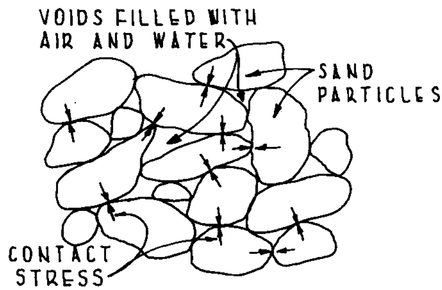


Figure 7-15 Typical Grain-to-Grain Contact in Sands

Sand and silt particles are obviously friable since they have been undergoing fragmentation for thousands of years to get to their present size. The theoretical point-to-point contact stress is infinitely large, so further chipping, crushing, cracking and rounding will occur whenever a foundation load is applied. As the grain sizes are thus reduced, the cracked and broken particles also undergo some shifting into a more dense state. As a result, the entire mass of sand undergoes a reduction in volume.

The fragmentation of the particles is a slow process, propagated from particle to particle. As the first few particles crumble or chip, the load shifts to other points. As these in turn crumble or chip, the load shifts to still other points. The resulting fragmentation can be quite slow, occurring over several weeks or months following the application of load.

Once a sand has undergone fragmentation, there is no rebound when the load is removed. The sand has undergone a permanent change in grain size, gradation and volume. The only recovery is the elastic rebound of the particles, which is very small in comparison to the fragmentation.

There is no standard test for fragmentation in sands comparable to the consolidation test for clays. A means to compute settlements in granular soils having either or both a silt component and a sand component is presented in the next chapter. The method is semiempirical and, even though it has some shortcomings, it is theoretically sound. The method does, however, require the use of empirical constants that cannot be verified and which can sometimes be badly in error.

Distortion settlements, or large-area settlements, occur in sands as well as in clays. In clays, the distortion settlements were found to be due to extrusion of porewater outward, away from the higher pressure areas at the center of the load. In sands, the distortion settlements are due to actual movement of the particles of sand.

As indicated in Fig. 7-16, there is less confining pressure at the edges of a loaded area and the particles of sand are more free to move. Consequently, some of the particles at the edges of the loaded area can be pushed outward, allowing a higher amount of settlement at the edges of the loaded area than at the center. This effect more than offsets the effects of fragmentation at the center of the area, where pressures (and confinement) are highest. The end result is a cupping downward at the edges of the loaded area, as shown.

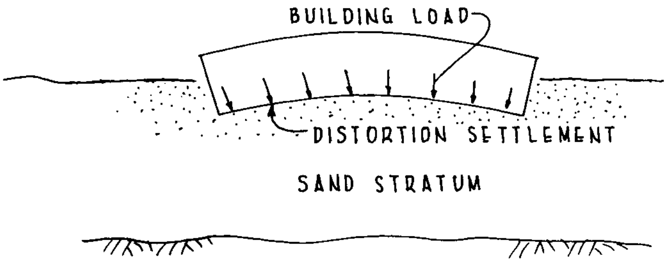


Figure 7-16 Distortion Settlements in Sands

As in clays, distortion settlements of small buildings on sand are not a major concern. Such settlements usually become a consideration only where the building is quite long, several hundred feet or more.

Elastic deformation on sand is a type of deformation that has no counterpart (or very little) in clays. Elastic deformations occur immediately upon application and release of load. Like distortion settlements, they affect the entire building footprint and are not a major source of unequal settlements between footings. Elastic deformations in soils are usually of little consequence in foundation design and are usually ignored.

Review Questions

7.1 What does the term "adsorbed" mean?

7.2 Describe a "flocculant" layup of fine soil particles.

7.3 What happens to a sensitive clay when it is remolded?

7.4 What type of load-settlement properties will a sensitive clay exhibit?

7..5 Define a "normally consolidated" clay.

7.6 How would a normally consolidated clay sustain a short-term impact Load?

7.7 How can a clay be identified from its *in situ* water content as being normally consolidated?

7.8 What is meant by "degree" of consolidation?

7.9 How is the amount of settlement related to the degree of consolidation in a clay soil?

7.10 Define an "overconsolidated" clay.

7.11 How would an overconsolidated clay sustain a short-term impact load?

7.12 How can a clay be identified from its *in situ* water content as being overconsolidated?

7.13 What is a desiccated clay?

7.14 In soils, what is the equivalent of a stress-strain curve?

7.15 In the time-settlement curve developed from the data of a consolidation test, how is the point of 100% consolidation determined?

7.16 In the time-settlement curve developed from the data of a consolidation test, how is the initial void ratio determined?

7.17 In the consolidation curve of a clay, how is the slope of the reload-rebound curve established?

7.18 Using the consolidation curve of a clay, how does one determine whether the clay is normally consolidated or overconsolidated?

7.19 For a given increment of pressure Δp, give the time-consolidation relationship for a clay soil at two different times during consolidation under this increment of load.

7.20 How can the relationship of question 7.19 be extended into some other increment of pressure Δp?

7.21 During the same increment of pressure Δp, how are degree of consolidation, settlement and porewater travel distance mathematically inter related?

7.22 During the same increment of pressure Δp, how are degree of consolidation, settlement and porewater travel distance mathematically related to time?

7.23 Compare distortion settlement in clays to distortion settlement in sands.

7.24 Describe fragmentation settlement in sands.

7.25 When a clay is loaded and allowed to undergo consolidation under the load, how much rebound can be expected under a short-term release of the load?

7.26 When a sand is loaded and allowed or forced) to undergo fragmentation under the load, how much rebound can be expected under a short-term release of the load?

OUTSIDE PROBLEMS

7-1 A laboratory sample of saturated clay is subjected to an abrupt increase in pressure of 3000 psf in a laboratory test. What is the porewater pressure at the beginning of the test and at 30% consolidation?

7-2 A laboratory sample of a saturated clay is subjected to a pressure of 4000 psf. At the end of 12 minutes, the porewater has dropped to 1500 psf and the settlement is measured at 0.02 inch.

1) Estimate the total settlement that can be expected at the end of the test.

2) Estimate how much time it will take to reach the end of the test.

7-3 A clay stratum 18 feet thick in a building site is overlain and underlain by strata of more permeable material. A laboratory sample of this clay, $^3/_4$ inch thick, drained two sides, reached 100% consolidation in 177 seconds under an increase in pressure of 3000 psf. Estimate how long it will take for the field stratum to reach 100% settlement at the same increase in pressure.

7-4 A consolidation sample 3/4 inch thick, drained two sides, reached 100% consolidation in 198 seconds under a particular increase in pressure. The actual field stratum from which the sample was taken is 7 feet thick, overlain by sand and underlain by an impermeable shale. Estimate how long it will take for the field stratum to reach 60% of its expected total settlement under this same increase in pressure.

7-5 A field stratum of clay is expected to reach a total settlement of 4.6 inches under a particular increase in pressure. At the end of 86 days, the actual settlement under this increase in pressure is measured and found to be 2.8 inches. Estimate the time required for the remaining settlement to occur.

7-6 The load on a footing 10 feet square founded 4 feet deep in a surface stratum of saturated clay 36 feet thick is increased abruptly by 112 kips. Porewater pressures under the footing are monitored by two pressure gages, the first located at the centerline of the footing just below the founding line and the second located 5 feet directly below the first. At the end of 39 days, the total settlement of the footing is measured and found to be 1.42 inches. At that time, the increase in porewater pressure at the level of the founding line is 305 psf.

1) Sketch the strata, showing the footing, the gages and the Boussinesq pressure bulb.
2) Estimate how much more settlement can be expected.
3) Estimate how long it will take for the remainder of the settlement to occur.

7-7 For the conditions of Problem 7-6, estimate the increase in porewater pressure at the lower pressure gage at the time the load is applied and at the end of 39 days. Recognizing that the time for 100% consolidation to occur at the lower gage will not be the same as that for the upper gage, determine some means to estimate roughly the amount of time required for 100% consolidation to occur at the lower gage.

7-8 A surface stratum of clay is 11 feet thick and is underlain by a more permeable soil. An embankment is to be placed over this clay which will produce an increase in pressure of 1000 psf in a relatively short period of time. A consolidation test is performed on a sample of this clay $^3/_4$ inch thick, drained two sides, under a pressure increase of 1000 psf, 100% consolidation was reached in 266 secs. How long will it take for 66% of the field settlement to occur? For 100% of the field settlement to occur?

PART III

DESIGN OF SHALLOW FOUNDATIONS ON A SOIL MASS

Chapter 9

EFFECTS OF SOIL-STRUCTURE INTERACTION*

Summary of Allowable Soil Pressures

The calculations for pressures and settlements in Chapters 6, 7 and 8 form a rather bewildering assortment of formulas, coefficients, correction factors and test results. It is well at this point to extract and summarize the proper equations and procedures that are used to determine the three basic design pressures p_a, p_a', and p_a''. Tables 9.1 and 9.2 contain such a summary of the required formulas and coefficients, along with an indication of the type of soil and the load combinations to which the equations apply.

The methods of Chapter 6 are used to establish the allowable strength of the soil both with and without lateral loads. The resulting pressures p_a and p_a' listed in Table 9-1 are based entirely on the strength of the soil, with no consideration given to settlements. The only difference between p_a and p_a' is that p_a' also includes a correction for an inclined load due to wind or earthquake shears.

It was noted earlier that when the methods of Chapter 6 are used in design, it is customary to ignore the cohesion c in soils classified as gravels, sands or low plasticity silts (ML). Similarly, it is customary to take the angle of internal friction ϕ to be zero in soils classified as clays or high plasticity silts (MH). The reliability of a friction component in in a clay soil or a cohesion component in a sand is sometimes questionable; they are usually ignored.

For settlements, the methods of Chapter 8 are used to determine the pressure pa'' at which settlements will not exceed a prescribed limit, usually one inch. The equations for the calculation of p_a'' are those listed in Table 9-2.

The pressure p_a'' listed in Table 9-2 is based entirely on settlement characteristics, with no consideration given to strength. On occasion, the settlement p_a'' may have to be estimated using empirical values; typical empirical values are presented in the following section.

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* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

Table 9-1 Calculation Guide for Determination of Design Pressures Based on Strength

CALCULATED ALLOWABLE SOIL PRESSURES BASED ON STRENGTH

Allowable pressure p_a under a maximum gravity load of DL + 100%LL

On sands $$p_a = \frac{1}{FS}\left[\tfrac{1}{2}\gamma B N_\gamma + \gamma D_f N_q\right]$$

To be corrected for shape, depth of founding and water table.

On all clays $$p_a = \frac{1}{FS}\left[\gamma D_f N_q + c N_c\right]$$

To be corrected for shape and depth of founding.

Allowable pressure p_a' for combined vertical and lateral load with load combination DL + 0.75(LL + W) or DL + 0.75(LL + E/1.4), where W and E/1.4 are the vertical loads, if any, induced by wind or earthquake.

The structural design must limit drift to 0.005 radians

On sands $$p_a' = \frac{1}{FS}\left[\tfrac{1}{2}\gamma B N_\gamma + \gamma D_f N_q\right]$$

To be corrected for shape, depth of founding, water table and inclined loads

On all clays $$p_a' = \frac{1}{FS}\left[\gamma D_f N_q + c N_c\right]$$

To be corrected for shape, depth of founding and inclined loads

Table 9-2 Calculation Guide for Determination of Design Pressures Based on Settlements

CALCULATED ALLOWABLE PRESSURES BASED ON SETTLEMENTS

Allowable pressure $\Delta p = p_a''$ for one inch settlement under a sustained gravity load of DL + 50%LL.

<u>On sands</u> $\quad S = \frac{1}{12}\text{ ft.} = C_1 C_2 p_a'' \sum_{0}^{\frac{4B}{2B}} \frac{I_z}{E_z} \Delta z$, solve for p_a''

$$C_1 = 1 - 0.05 \frac{\gamma D_f}{p_a''} \quad \text{and} \quad C_2 = 1.2 + 0.2 \log_{10} t \text{ (years)}$$

<u>On normally consolidated clays</u>

$$S = \frac{C_C}{1+e_0} H \log_{10} \frac{p_0 + p_a''}{p_0}, \text{ solve for } p_a''$$

where $C_C = 0.009\,(w_L - 10)$

<u>On overconsolidated clays</u>

where $p_a'' \leq$ preconsolidation pressure p_{PRE}

$$S = \frac{C_{rr}}{1+e_0} H \log_{10} \frac{p_0 + p_a''}{p_0} \quad \text{solve for } p_a''$$

$$\text{where } C_{rr} = \frac{e_0 - e_{PRE}}{\log_{10}\left(\frac{p_{PRE}}{p_0}\right)}$$

and e_{PRE} is the void ratio at p_{PRE}

<u>On overconsolidated clays</u>

where $p_a'' >$ preconsolidation pressure p_{PRE}

$$S = \frac{C_{rr}}{1+e_0} H \log_{10} \frac{p_{PRE}}{p_0} + \frac{C_{vc}}{1+e_0} H \log_{10} \frac{p_0 + p_a''}{p_{PRE}}; \quad \text{solve for } p_a''$$

$$\text{where } C_{rr} = \frac{e_0 - e_{PRE}}{\log_{10}\left(\frac{p_{PRE}}{p_0}\right)} \quad \text{and} \quad C_{vc} = \frac{e_{PRE} - 0.42 e_0}{\log_{10}\left(\frac{p_{0.42}}{p_{PRE}}\right)}$$

and e_{PRE} is the void ratio at p_{PRE}

and $p_{0.42}$ is the pressure at $0.42 e_0$

Estimated Pressure-settlement Relationships

The low level of accuracy in the calculation of pressure-settlement relationships in soils has been noted repeatedly. Whenever such limitations appear in the theoretical methods, it is common practice in foundation design to place more reliance on tried and proven empirical methods than to use questionable calculations. Such empirical methods have evolved over the years concerning the question of pressure-settlement relationships in soils.

Probably the most reliable estimates relating bearing pressures to settlements are those that have withstood the test of time. Decades ago, Terzaghi and Peck[38], listed their personal estimates of pressure-settlement relations in both sands and clays. Over the ensuing years, other responsible soils engineers have refined these estimates into a relatively dependable set of values.

Admittedly, the empirical estimates apply best to large brackets of load systems and soil types, but they form a reassuring reference for "judgement calls" when one is estimating settlements. As a basis for comparison of calculated results, an updated summary of Terzaghi and Peck's estimates is presented in this section.

Settlements in clays have long been linked to the allowable increase in bearing pressure and the resulting consolidation. Long before the advent of theoretical soil mechanics, there was a belief that an allowable bearing pressure for clay soils could be established such that settlements would not be a problem. There was no attempt to define the relationship between the increase in pressure and the induced settlement. It was only presumed that if soil pressures were limited to some unknown but moderate fraction of the ultimate strength, the whole problem of settlements in clay would go away.

Over the years, it has become generally accepted that this long-sought factor of safety can be taken as 3. It was found that if clay soils are designed with a factor of safety of 3 to bearing capacity failure, the corresponding settlements would be within acceptable levels. This presumption has withstood the test of time and continues to be a useful guide; it is in fact widely accepted in today's engineering practice. Although the actual magnitude of the settlements is not a part of the presumption, it can be assumed that the differential settlement between two footings will not be more than about $^3/_4$ inch[31].

Pressure-settlement relationships in sand are not quite as simple to estimate as the foregoing relationships for clays. Settlements in sand are dependent on many variables, including gradation, grain size, grain shape and grain hardness as well as the overall size of the footing and its depth of founding. Unlike clays, a simple limit on the factor of safety does not work for sands.

For design, the limiting value of soil pressure that will limit settlements to about 1 inch is largely a "judgement call", derived from a combination of experience, testing and judgement. A summary of such soil pressures is given in Table 9-3.

Table 9-3 Estimated Pressure p_a'' on Soils to Produce Roughly 1 inch of Settlement

(Corrected as suggested by Das[12])

Allowable pressure $p_a'' = \Delta p$ for one inch settlement under a sustained gravity load of DL + 50%LL.

On sands

Estimated pressure p_a'' on sands to produce roughly 1 inch of settlement without regard to strength limitations

Angle of internal friction ϕ	Contact pressure p_a'' psf	Angle of internal friction ϕ	Contact pressure p_a'' psf
28	1125	36	10125
29	2250	37	11250
30	3375	38	12375
31	4500	39	13500
32	5625	40	14625
33	6750	41	15750
34	7875	42	16875
35	9000	43	18000

On all clays

Estimated pressure p_a'' on clays to produce roughly 1 inch of settlement at one third ultimate strength

$$p_a'' = \frac{1}{3}\left[\gamma D_f N_q + cN_c\right]$$

To be corrected for shape and depth of founding.

The simplicity of Table 9-3 is due in large part to the nature of settlements in sand. Settlements in sand are affected by the gradation of the sand, by the grain size and distribution, by grain hardness and by grain shape. In general, these are identically the same factors which determine the angle of internal friction of the sand. It should be expected that settlements in sand would be essentially proportional to the angle of internal friction, as indicated in Table 9-3.

Effects of Structural Design on Foundation Design

Given the loads acting on a footing from above and the allowable pressure that can be supported by the soil below, calculations for the size of the contact area of a footing would seem to be very straightforward. The interaction between soil and structure, however, can have a significant influence on the final selection of the size of the footing. The effects of such soil-structure interaction must be accommodated in the design.

One such interactive effect comes from rotations of the footing due to structural deformations in the building above. Obviously, the effects of such rotations must somehow be included in the design. In the method developed in this chapter, it will be seen that the effects of these rotations (when they occur) can be included by using only a modified vertical load; the actual moments and rotations need not be found. The final selection of the required footing area is then reduced to a simple calculation involving only a vertical load and an allowable pressure.

In recognition of the general level of accuracy in soils, simplified statically determinate approaches are used throughout the subsequent discussions. The final calculations use pressures rather than settlements since the prediction of the magnitude of settlements is in itself questionable. However, the thoughtful comparison of two calculated settlements can often provide insight into the relative movements of the two foundations: such comparisons are utilized repeatedly in the following discussions.

Throughout all calculations involving soils, only working levels of load are used. Design concepts involving ultimate loads are not yet applied to soils.

Footings With Vertical Load Only

The simplest case for footing design is that of a footing carrying vertical load only. Moments are prevented by providing a hinge between the column and footing, as shown in the example footing of Fig. 9-1. The structural analysis of the building above must, of course, include the effects of the hinged column base.

With no moment on the footing, the bearing pressure is assumed to be uniform, as shown in Fig. 9-1. The required contact area for the footing is found very simply:

$$A = \frac{P}{p}$$

where P is the vertical load
A is the contact area
p is the allowable bearing pressure

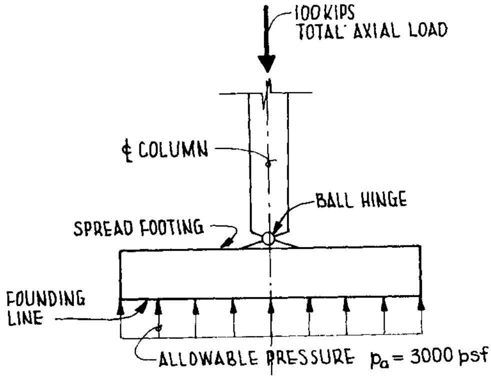


Figure 9-1 Direct loads on footings.

In the example of Fig. 9-1, the indicated column load is 100 kips and the allowable bearing pressure is 3000 psf. The required contact area is found to be:

$$A = \frac{P}{p} = \frac{100000}{3000} = 33 \text{ ft}^2$$

A footing 6 ft x 6 ft would satisfy the requirements. In this case, it it is not stated (nor does it matter) whether the bearing pressure is limited by strength criteria or settlement criteria.

The use of hinged column bases as shown in Fig. 9-1 is much preferred insofar as foundation design is concerned. It has the advantage that the system is statically determinate and is therefore predictable. There are no statically indeterminate deformations that may or may not develop as presumed.

Rotations of a footing are a very common feature, however, occurring whenever the footing is fixed to the base of the column. The effects of column moments and the rotations of the footing produced by these moments are considered next.

Effects of Column Moments on Footing Rotations

The reason for limiting the settlement of a footing is to prevent damage to the structure above. The only settlement that is of interest, therefore, is the settlement at the centerline of the column or bearing wall. The magnitude of the settlements elsewhere on the footing is not usually of interest.

A typical footing with a concentric column rigidly attached to its footing is shown in Fig. 9-2. Exaggerated rotation of the footing due to a column moment is shown in phantom. A strip footing supporting a bearing wall would show a similar response.

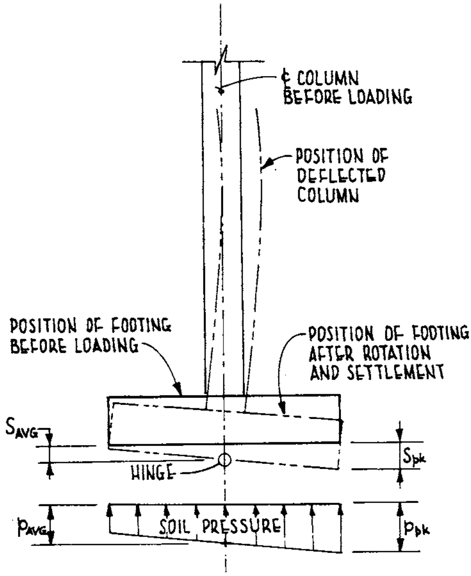


Figure 9-2 Typical Column and Footing

The trapezoidal pressure diagram of Fig. 9-2 is a result of bending in the column which in turn produces rotation of the footing. A moment's reflection will affirm that the average settlement as well as the average pressure will occur at the centerline of the symmetrical footing. The rotation due to the column moment has no effect on the average pressure p_{AVG}, but it does have an effect on the peak pressure p_{pk}. Where strength of the soil is the limiting condition, it is the peak pressure p_{pk} that must not exceed the allowable pressure p_a or p_a'.

Alternatively, where settlement of the column is the limiting factor, it is the average settlement of the footing, S_{AVG}, that must be limited (Fig. 9-2). At the centerline, it is noted that settlement of the column will be proportional to the average pressure p_{AVG}. Consequently, if the settlement of the column is not to exceed 1 in., it is the *average* pressure p_{AVG} that must not exceed the allowable bearing pressure p_a''.

The foregoing conclusions for concentrically loaded footings are summarized; they will be used repeatedly in subsequent discussions.

- Average pressure and average settlement occur at the centerline (centroid) of a footing.
- Moments in the column have no effect on the average pressure or average settlement.
- The peak pressure occurs at the edge of the footing and is caused by rotations of the footing.

[As a point of realistic structural design, the hinge point (and thereby the effective length) of a ground-floor column should be taken at the centroid of the contact surface of its footing as shown in Fig. 9-2, rather than at the top.]

Effects of Footing Rotations on Soil Pressures

A typical concentric column and footing are shown in Fig. 9-3. Exaggerated rotation of the footing and the effect on the soil pressures are shown. The formulas shown for computing the magnitudes of moments and rotation are given in standard reference textbooks[15].

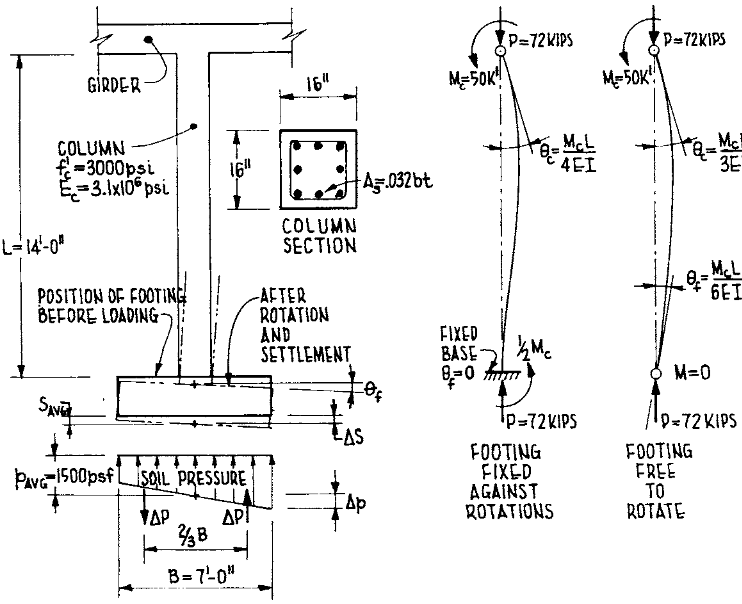


Figure 9-3 Typical Footing Rotations

The following symbols appear in Fig. 9-3:

f_c' is the ultimate strength of concrete.
E_c is the modulus of elasticity of concrete.
I is the moment of inertia of the cross section.
b is the width of the cross section.
t is the depth of the cross section.

If the footing is not allowed to rotate at all, the moment at the base of the column will be $\frac{1}{2}M_c$, as shown. Alternatively, if the footing is allowed to rotate freely, the moment at the base of the column will be zero, with a rotation angle θ_f as shown. It should be evident that the amount of rotation that the footing is allowed to undergo (corresponding to the degree of restraint) will determine the magnitude of the moment that will exist at the base of the column.

Refer again to Fig. 9-3, where the soil pressures and their corresponding settlements are shown on the soil pressure diagram. It is assumed that within the range of pressures that may occur, the settlement is directly proportional to pressure.

The large moment at the top of the column in Fig. 9-3 is likely due to wind or earthquake loads; gravity loads do not usually produce such large column moments. The rotation q_C at the top of the column is readily computed using the approximation $I = bh^3/12 = 5461$ in. and $E_C = 3{,}100{,}000$ psi:

$$\theta_c = \frac{M_c L}{4E_c I} = \frac{50{,}000 \times 12 \times 14 \times 12}{4 \times 3{,}100{,}000 \times 5461} = 0.0015 \text{ radians}$$

For the sake of reference, the stress f_C in the concrete at the top of the column was found in an outside calculation using conventional methods[15],

$$f_C = 827 \text{ psi}$$

This stress is in a comfortable range insofar as column stress is concerned.

To see the effect if the footing rotation is so small it is negligible, it is assumed that the base of the column is fixed to the footing and that the footing is highly resistant to rotations. The maximum value of moment that could occur at the base of the column is then

$$M_f = \frac{M_C}{2} = 25 \text{ kip-ft}$$

For this moment, the resultant trapezoidal soil pressure diagram is shown in Fig. 9-4, where the couple formed by the force ΔP must oppose the applied moment M_f.

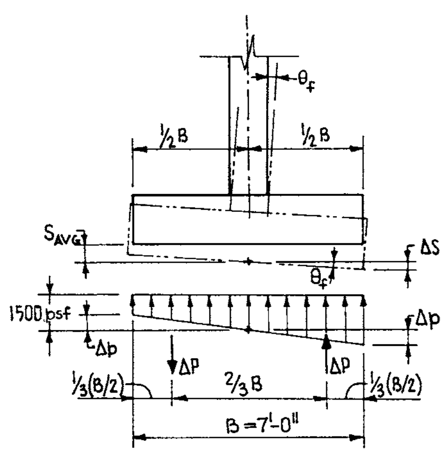


Figure 9-4 Footing rotations, first case.

The solution for ΔP shown in Fig. 9-4 is obtained from the definition of a couple,

$$M_f = \Delta P\left(\frac{2B}{3}\right); \quad 25000 \times 12 = \Delta P(7 \times 12 \times \tfrac{2}{3}); \quad \Delta P = 5360 \text{ lbs}$$

The solution for the increase in pressure ΔP is obtained by simple statics,

$$\Delta P = (\tfrac{1}{2}\Delta p)(\tfrac{1}{2}B^2); \quad 5360 = (\tfrac{1}{2}\Delta p)\,\tfrac{1}{2}(7 \text{ x } 7); \quad \Delta p = 438 \text{ psf}$$

The change in settlement ΔS at the edge corresponding to this change in pressure is found by simple ratios, where the indicated pressure of 1500 psf is assumed to produce a settlement in the range of 1 in.:

$$\frac{1}{1500} = \frac{\Delta S}{438}; \quad \Delta S = 0.3 \text{ in.}$$

The corresponding footing rotation due to the moment of 25 kip•ft is then

$$\theta_f = \frac{\Delta S}{\frac{1}{2}B} = \frac{0.3}{42} = 0.007 \text{ radians}$$

Rather than being nearly zero as assumed, this rotation is seen to be some $4\tfrac{1}{2}$ times the amount of elastic rotation (0.0015 rad) already computed at the top of the column. This rotation would cause a stress of more than 3800 psi, exceeding the ultimate strength of the concrete. It would also produce impossible conditions for compatibility of deformations at the base of the column.

It is concluded, therefore, that the assumption that produced this result (that the footing and soil could provide a relatively rigid base) was grossly incorrect. That assumption will therefore be discarded and the other extreme will be examined. It will now be assumed that the footing still provides adequate support for its vertical load but that it undergoes small rotations relatively freely, rather like a raft floating in water.

For this second case, the base of the column is assumed to be fixed to the footing as before, but it is now assumed that the soil offers very little resistance to rotations. The result is that the footing rotates through the angle θ_f as shown in Fig. 9-5, again producing a trapezoidal pressure-settlement diagram as shown.

The angle θ_f is computed as before,

$$\theta_f = \frac{M_c L}{6E_c I} = \frac{50{,}000 \times 12 \times 14 \times 12}{6 \times 3{,}100{,}000 \times 5461} = 0.0010 \text{ radians}$$

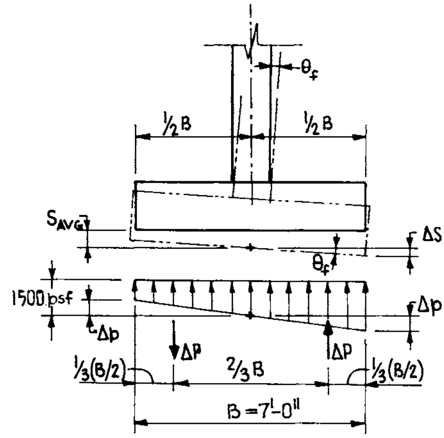


Figure 9-5 Footing rotations, second case.

The settlement ΔS at the edge of the footing due to a rotation of 0.001 radians is readily computed:

$$\Delta S = \left(\frac{B}{2}\right)\theta_f = 42 \times 0.0010 = 0.042 \text{ in.}$$

The increase in pressure at the edge is found by ratios, as before,

$$\frac{1}{1500} = \frac{1 + \Delta S}{1500 + \Delta p} = \frac{1.042}{1500 + \Delta p}; \qquad \Delta p = 63 \text{ psf}$$

The increase in pressure is thus found to be 4.2% above the average pressure of 1500 psf. Within the usual range of accuracy for soils, such an increase is negligible.

Consequently, it is concluded that the analysis of this second case is essentially correct as it stands. The first conclusion to be drawn from these calculations is that *this soil offers practically no resistance to rotations of a footing*, at least not within the usual range of rotations that can be tolerated in the concrete column.

The small resisting moment incurred by the rotation (the small error in the foregoing calculations) can be found by statics; this moment is shown in Fig. 9-5 as the couple ΔP and its arm, $^{2B}/_{3}$:

$$M_f = \Delta P\left(\frac{2B}{3}\right) = \tfrac{1}{2}(\Delta p)\frac{B}{2}B\left(\frac{2B}{3}\right) = \Delta p\frac{B^3}{6}$$

$$M_f = 63\left(\frac{7 \times 7 \times 7}{6}\right) = 3601 \text{ lb} \bullet \text{ft}$$

This moment is seen to be less than 15% of the 25 kip•ft moment that would be incurred if the base were truly fixed. It is again a small amount within the usual range of accuracy for soils. The second conclusion to be drawn here is that *no significant fixity of the column base has been provided by the isolated footing.*

The foregoing discussions demonstrate that for an isolated footing founded on a soil having a low-to-medium strength:

- The soil offers essentially no resistance to rotations of the footing.
- No significant fixity of a column base is provided.

The foregoing two conclusions are generalized in the next section and shown to be true for most of the smaller buildings and soil types encountered in routine practice. Since shallow foundations are generally suitable only for smaller buildings, the foregoing conclusions are thus true for most applications of shallow foundations.

Generalization of Effects of Rotations

The particular solution of the preceding section is repeated here for a general case. In developing this general case, it is assumed that the design conditions are at or near their most severe. A sketch of the general column and footing is shown in Fig. 9-6. Formulas for rotations due to flexure are given in standard references[15].

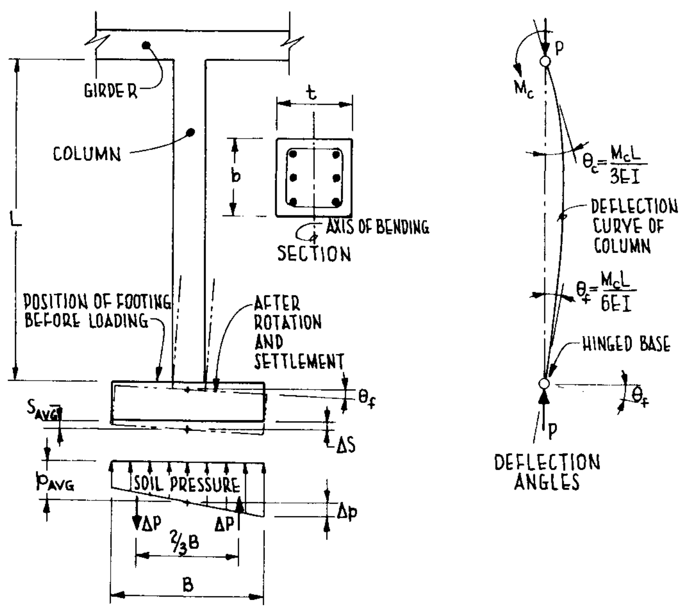


Figure 9-6 Typical column and footing.

The rotation at the base of the column in Fig. 9-6 is:

$$\theta_f = \frac{M_c L}{6 E_c I} \tag{9-1}$$

where M_c is the moment at the top of the column.

It is assumed that the ratio M_c/I in Eq.(9-1) is at its highest, where by the flexure formula, $f_c = M_c c/I$ and $c = kd$,

$$\frac{M_c}{I} = \frac{f_c}{kd} \tag{9-2}$$

For Eq.(9-2) it is assumed that f_c approaches $^1/_2 f_c'$ and that the depth of compression kd approaches $t/3$ for a lightly loaded concrete column. To obtain the maximum value for the ratio, f_c' is assumed to be quite large, 5000 psi, and the size t to be quite small, 12 in.

It is assumed that the maximum unsupported length L of a column is 16 ft for first-floor columns: the modulus of elasticity of concrete, E_C, is taken at $57{,}000\sqrt{f_c'}$ (ACI 318-83). The foregoing values are substituted into Eq.(9-1). The solution for the rotation of the column section is then:

$$\theta_f = \left(\frac{M_c}{I}\right)\left(\frac{L}{6E_c}\right) = \left(\frac{f_c}{kd}\right)\left(\frac{L}{6E_c}\right) = \frac{\frac{1}{2}\times 5000\times 16\times 12}{\frac{1}{3}\times 12\times 6\times 57{,}000\sqrt{5000}}$$

$$= 0.005 \text{ radians}$$

It is concluded that the highest reasonable value of rotation that can occur at the base of a concrete column is about 0.005 radians. (It should be recalled from Chapter 3 that the nominal "drift" angle for a frame line and its footings is also 0.005 rad, which is consistent with these findings.)

The soil pressure diagram of Fig. 9-6 reveals that for a general soil pressure p_{AVG}, at an average settlement in the range of 1 in., the ratio of pressures is

$$\frac{p_{AVG}}{1\text{ in.}} = \frac{p_{AVG} + \Delta p}{1\text{ in.} + \frac{1}{2}B\theta}, \quad \text{or,} \quad \frac{\Delta p}{p_{AVG}} = \frac{B\theta}{2}, \quad \text{B in inches} \tag{9-3}$$

The maximum rotation, $q_f = 0.005$ rad, is substituted into this equation, yielding

$$\frac{\Delta p}{p_{AVG}} = \frac{B}{400} \tag{9-4}$$

For small footings ($B = 24$ in.), the increase in soil pressure given by the foregoing relationship is about 6%. For large footings ($B = 100$ in.), the increase in soil pressure is found to be about 25%. (Note that an extremely long rectangular footing such as a strip footing under a shearwall does not fit into this estimate.)

The foregoing investigation indicates that for concrete structures, soil pressures due to moments on an isolated rectangular or square footing may increase a maximum of only 25% even under the most adverse combinations of load. For steel columns, a similar analysis shows that an increase of about 35% above the average pressure could occur, although several arbitrary choices have to be made in arriving at such a conclusion.

The degree of fixity at the base of the column can now be found (for use in moment distribution or slope-deflection analyses). An approximate column moment is first found from the flexure formula, where the approximate section modulus is $S_c = bt^2/6$, with $f_c = {}^1/_2 f_c'$ and $b = t$:

$$M_c = f_c S_c = \frac{\frac{1}{2} f_c' b t^2}{6} = \frac{f_c' t^3}{12} \tag{9-5}$$

The restoring moment of the footing is found from the pressures in Fig. 9-6:

$$M_R = \tfrac{1}{2}(\Delta p)\tfrac{1}{2}B^2\left(\frac{2B}{3}\right) = \Delta p\left(\frac{B^3}{6}\right) \tag{9-6}$$

The ratio M_R/M_C is then

$$\frac{M_R}{M_C} = 2\frac{\Delta p}{f_C'}\left(\frac{B}{t}\right)^3 \tag{9-7}$$

For Eq.(9-7), a nominal value of B/t is assumed to be 2.5, the highest soil pressure to be 5000 psf, and lowest value for f_C' to be 3000 psi. Hence

$$\frac{M_R}{M_C} = 2\left(\frac{0.25\times 5000/144}{3000}\right)2.5^3 = 0.09$$

The actual restraint of the footing is therefore less than 10% of the moment capacity of the column. (For those familiar with moment distribution, the maximum fixity that could exist is thus seen to be less than 20% of the base moment and could be considerably less.)

In summary, it is concluded that:

- When a concrete column is rigidly attached to its spread footing, the footing can never rotate far enough to produce major changes in soil pressure.
- At most, the magnitude of the soil pressure at the edge of the footing cannot reasonably be more than 25% above the average.
- The footing produces no reliable level of resistance to the elastic rotations of the column.
- There is no dependable degree of fixity of the column produced at its base.

Attachment of Columns to Footings

In view of the foregoing conclusions, it is apparent that there is rarely any benefit to be obtained by fixing a column (or bearing wall) to its spread footing. Superficially, it would seem that the best solution is simply to provide a hinged base for the column, thereby eliminating any potential moment on the footing. There would then be no possibility that the potential 25% peak could ever occur.

As indicated in Fig. 9-7, a hinged base for a steel column is probably the simplest and most economical solution in most circumstances. For concrete columns, however, it may be the simplest but not the most economical solution.

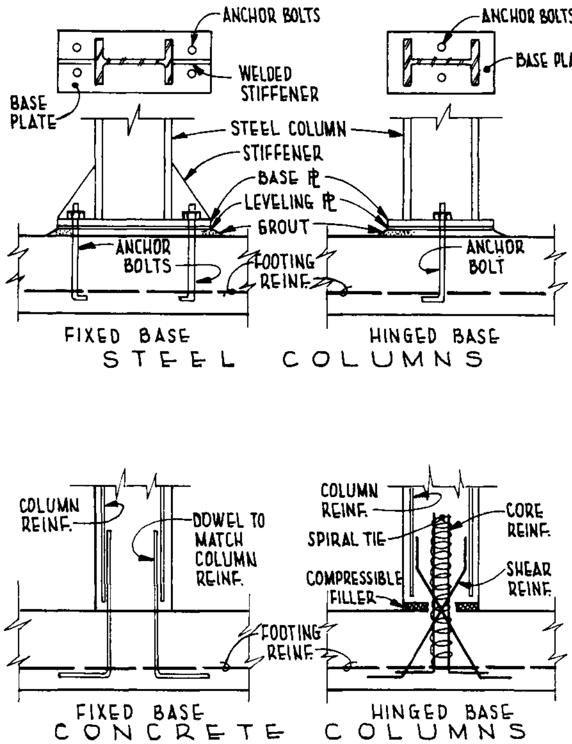


Figure 9-7 Typical column bases.

An examination of the typical details for column bases shown in Fig. 9-7 indicates that a fixed base for a steel column is considerably more elaborate than a hinged base. As a consequence, the fixed base will be more costly. Except in unusual circumstances, there is therefore little justification to fix a steel column to an isolated footing; a hinged base is much preferred.

For concrete, the opposite is true. The rather elaborate hinged base for a concrete column shown in Fig. 9-7 is considerably more difficult and costly to build than the simple dowels of a fixed base. As a consequence, it is common practice simply to dowel a concrete column (or bearing wall) to its footing and to suffer the consequences of the more complicated design calculations.

It should be recognized that the complications in the design calculations arise only when the limiting pressure is governed by strength, p_a or p_a'. When the limiting pressure is governed by settlements, p_a'', there is no concern about peak pressures at the edge of the footing; only the average pressure is of concern.

In addition, it should also be recognized that the complications arise only for spread footings or strip footings, which do not provide a moment resistant foundation. In contrast, grade beams inherently provide a moment-resistant foundation along their entire length. Where feasible, providing a grade beam foundation and fixing the colums to the grade beam eliminates this problem of peak pressures in the underlying soil.

An alternative to grade beams is the narrow moment-resistant "tie beam" shown in Fig. 9-8. The tie beam spans between the columns and may actually rest on the top of the footing. The tie beam can be designed to relieve the footing of any column moments. It also doubles as the cutoff wall for the floor slab and provides support for the wall above.

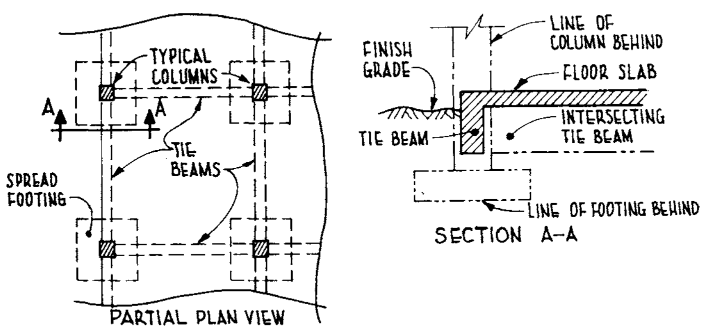


Figure 9-8 Typical Tie Beams

The following four items summarize the conclusions reached up to this point:

1. In settlement calculations, the strength of the soil may be assumed to be adequate to take any potential increases in pressure at the edges of the footing due to rotations. Spread footings may therefore be designed for the average soil pressure (vertical load ÷ contact area) regardless whether the columns (or bearing walls) are fixed or hinged to their footings.
2. In strength calculations when column bases are hinged, no peak pressure occurs in the soil; the soil pressure under the footing is uniformly

distributed. The spread footings may therefore be designed for an average soil pressure (vertical load ÷ contact area) equal to this uniform pressure.

3. In strength calculations when column bases are fixed (very common in concrete buildings), a peak soil pressure will occur at the edge of the footing. The design should provide for this potential 25% increase above the average pressure.
4. When grade beams or tie beams are used at the foundation, the column moments are taken by these beams and the moments cannot cause any peaks in soil pressures. The grade beams or spread footings may therefore be designed for the average soil pressure (vertical load ÷ contact area).

In three of the foregoing four cases, there is no peak pressure. The only exception is seen to be in item 3, where strength governs the design and column bases are fixed to the footing. Except for this case, all footings could be designed using only the average soil pressure (vertical load ÷ contact area), ignoring moments and rotations of the footings. The incidence of item 3, however, is very common; it includes the footings that support shearwalls when the shearwall undergoes a drift of 0.005 radians.

Obviously, some simple method to transform item 3 is needed such that the same simple design procedure may be used in all cases. There are, of course, many such methods that might be developed. One such method is presented in the following section.

Peak Pressure In Strength Calculations

It has been seen that fixing the base of a column to its spread footing can cause an increase in the soil pressure of uncertain magnitude at the edge of the footing. At its worst, however, such an increase will be less than about 25% above the average soil pressure for concrete columns, slightly higher for steel columns. In view of this, it is desirable to develop a simplified method of design that uses only the average soil pressure; the uncertain peak pressure can then be accommodated without having to make a special analysis for it.

Such a method is readily possible. The applied vertical load is simply magnified by a suitable multiplier. The footing is then designed to limit the average soil pressure produced under this magnified load. Under the lesser actual load, the peak pressure will then always be less than the allowable pressure; its actual magnitude will no longer be of any concern. A comparison of the two cases is shown in Fig. 9-9.

The problem, of course, lies in selecting a suitable multiplier. It must produce an average pressure under the magnified load that is only slightly higher than the peak pressure under the actual load. Relative levels for these pressures are compared in the pressure diagrams of Fig. 9-9.

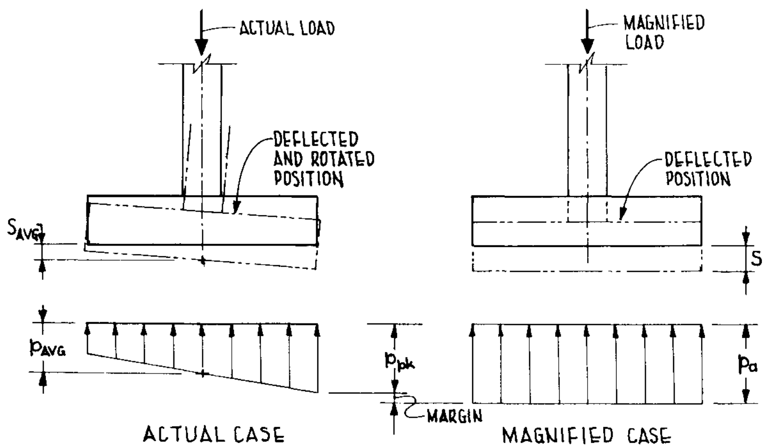


Figure 9-9 Actual load and magnified load.

A 25% increase would produce the desired result for concrete buildings. For steel buildings, the peak pressure may be somewhat higher, adding as much as another 10%. It is noted that both of these values are based on the incidence of all the worst-case conditions occurring at the same time, a rather remote possibility. In this book, an increase of 25% is adopted, to apply to all structures.

The design procedure to account for the type of attachment thus becomes quite simple:

- When a footing is *fixed* to the structural member above and load is limited by *strength* of the soil rather than by settlements, the maximum vertical load that may occur at the footing is multiplied by a magnifier of 1.25:

$$P_{MAG} = 1.25(\text{DL} + 100\%\text{LL}),$$

$$\text{or, } P_{MAG} = 1.25[\text{DL} + 0.75(\text{LL} + \text{W or E})]$$

- The size of the contact area is determined for these magnified vertical loads, using the specified allowable soil pressure p_a or p_a' as appropriate.

For all other cases, whether fixed or hinged, whether limited by strength or settlement, the design loads are unchanged.

Applications in Selecting Final Footing Sizes

Some examples will illustrate the selection of footing sizes to suit the soil conditions, the load conditions and the structural configuration.

Example 9-1 Final selection of footing sizes

Given : First interior footing D2 in the steel rigid frame line of Examples 6-7 and 8-11, as shown in the sketch.

All footings are hinged to their columns.

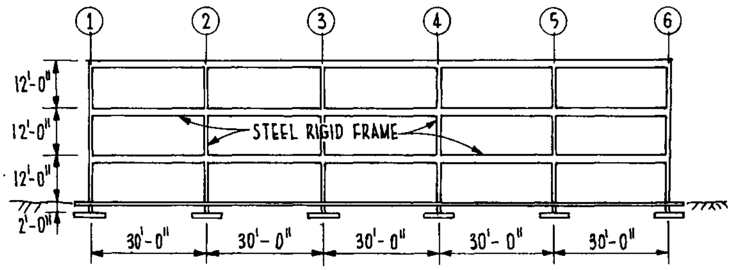
Reference footing is 6 ft square with depth of founding of 3 ft.

Soil is a poorly graded sand, SP.

h_{LAT} = 21.9 ft. for wind loads
h_{LAT} = 26.6 ft. for earthquake loads

To find: 1) Required size of the footing

2) Check adequacy in frictional resistance



FRAME LINE D

Solution:

Determine the footing sizes first

From Example 6-7,

DL = 70 kips, LL = 50 kip
V = 55 kips per frame line (wind)
V = 60 kips per frame line (earthquake)

For hinged column bases, no magnification of loads is used.

Max. gravity load	= DL + LL = 70 + 50 = 120 kips
Max. combined load	= DL + 0.75(LL + W), or
	= DL + 0.75(LL + $E_{ELASTIC}$
	= 70 + 0.75(50 + 0) = 108 kips
Max. sustained load	= DL + 50%LL = 70 + 25 = 95 kips

For the reference footing 6 ft square with a depth of founding of 3 ft, allowable pressures p_a and p_a' are calculated in Examples 6-7 and allowable pressure p_a'' is calculated in Example 8-8. The pressures are repeated here for immediate reference.

p_a = 3860 psf for maximum gravity loading,
P_{ref} = 3860 x 6 x 6 = 139 kips

p_a' = 3010 psf with wind,
P_{ref} = 3010 x 6 x 6 = 108 kips

p_a' = 2950 psf with earthquake,
P_{ref} = 3270 x 6 x 6 = 106 kips

p_a'' = 2300 psf to produce a settlement of 1 inch,
P_{ref} = 2300 x 6 x 6 = 83 kips

Selection of footing size for footing D2 on a sand, using Equation (6-23):

For maximum gravity load with P_{ref} = 139 kips,

$$B_1 = \sqrt[3]{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt[3]{\frac{120}{139}} \times 6 = 5.71 \text{ ft.}$$

For maximum combined load with P_{ref} = 108 kips for wind and 106 kips for earthquake,

$$B_1 = \sqrt[3]{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt[3]{\frac{108}{108}} \times 6 = 6.00 \text{ ft. with wind}$$

$$B_1 = \sqrt[3]{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt[3]{\frac{108}{106}} \times 6 = 6.04 \text{ ft. with earthquake}$$

For maximum sustained load with P_{ref} = 83 kips, using Equation (8-6)

$$B_1 = \frac{P_1}{P_{ref}} B_{ref} = \frac{95}{83} \times 6 = 6.87 \text{ ft. with settlement } = 1 \text{ in.}$$

<u>USE SQUARE FOOTING, B = 7 ft 0 in.</u>

Check for frictional resistance

Review for frictional resistance:

Maximum lateral load = V/n bays = 60/5 = 12 kips

Minimum friction force F that can be developed:

$F = \mu(\text{DL}) = 0.3 \times 70 = 21$ kips > 12 kips (O.K.)

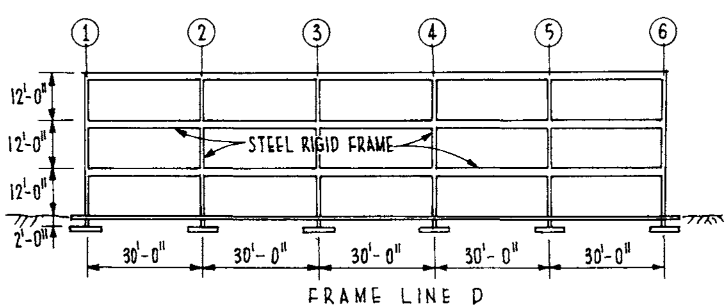
Example 9-2 Continuation of Example 9-1.

Given : End footing D1 in the steel rigid frame line of Examples 6-8 and 8-11, as shown in the sketch.

All footings are hinged to their columns

Reference footing is 6 ft square, with depth of founding of 3 ft.

Soil is a poorly graded sand, SP.



To find: 1) Required size of the footing

2) Check for adequacy of frictional resistance and restoring moment

Solution:

Determine footing sizes first

Loads on footing D1 are given in Examples 6-7 and 6-8.

DL = 55 kips, LL = 40 kips

V = 55 kips per frame line due to wind

V = 60 kips per frame line due to earthquake

W = Vh_{LAT}/L = 55 x 21.9/150 = 8 kips

E = Vh_{LAT}/L = 60 x 26.6/150 = 11 kips (elastic)

For hinged column bases, no magnification of loads is used.

Max. gravity load = DL + LL = 55 + 40 = 95 kips

Max. combined load = DL + 0.75(LL + W or $E_{ELASTIC}$)

= 55 + 0.75(40 + 8) = 91 kips

= 55 + 0.75(40 + 11) = 93 kips

Max. sustained load = DL + 50%LL = 55 + 20 = 75 kips

For the reference footing 6 ft square with a depth of founding of 3 ft, allowable pressures p_a and p_a' are calculated in Examples 6-8 and allowable pressure p_a'' is calculated in Example 8-8. The pressures are repeated here for immediate reference:

p_a = 3860 psf for maximum gravity loading,

P_{ref} = 3860 x 6 x 6 = 139 kips

p_a' = 3335 psf with wind,

P_{ref} = 3335 x 6 x 6 = 120 kips

p_a' = 3270 psf with earthquake,

P_{ref} = 3270 x 6 x 6 = 118 kips

p_a'' = 2300 psf to produce a settlement of 1 inch,

P_{ref} = 2300 x 6 x 6 = 83 kips

Selection of footing size for footing D1 on a sand:

For maximum gravity load with P_{ref} = 139 kips using Equation (6-23):

$$B_1 = \sqrt[3]{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt[3]{\frac{95}{139}} \times 6 = 5.28 \text{ ft.}$$

For maximum combined load with P_{ref} = 120 kips for wind and 118 kips for earthquake,

$$B_1 = \sqrt[3]{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt[3]{\frac{91}{120}} \times 6 = 5.47 \text{ ft. with wind}$$

$$B_1 = \sqrt[3]{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt[3]{\frac{93}{118}} \times 6 = 5.54 \text{ ft. with earthquake}$$

For maximum sustained load with P_{ref} = 83 kips using Equation (8-6):

$$B_1 = \frac{P_1}{P_{ref}} B_{ref} = \frac{75}{83} \times 6 = 5.42 \text{ ft. with settlement } = 1 \text{ in.}$$

<u>USE SQUARE FOOTING, B = 5 ft 7 in.</u>

Check frictional resistance and restoring moment

Determine frictional resistance:

Maximum lateral load = $V_b/2n$ bays = 60/10 = 6 kips

Minimum friction force F that can be developed:

$F = \mu$(DL) = 0.3 x 55 = 17 kips > 6 kips (O.K.)

The restoring moment on a rigid frame is always identical to the overturning moment, $M_{OV} = M_R = Vh_{LAT}$, where h_{LAT} is the height to the center of lateral loads.

For this structure, the restoring moment at the two end bays must be developed by the three girders:

For wind, M_{OV} = 55 x 21.9/2 = 600 kip•ft.
M_R = 8 x 150/2 = 600 kip•ft.

For earthquake, M_{OV} = 60 x 26.6/2 = 800 kip•ft.
M_R = 11 x 150/2 = 800 kip•ft.

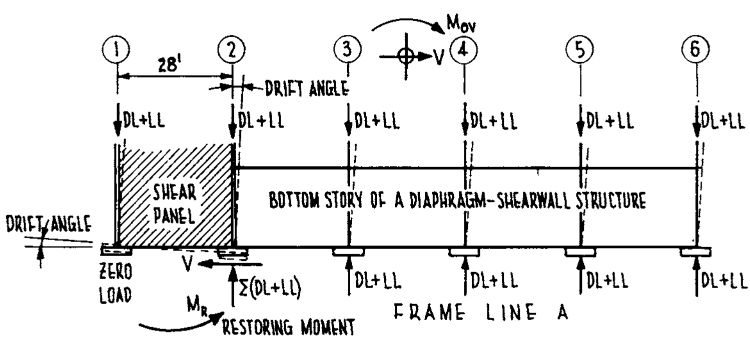
Example 9-3 Final selection of footing sizes

Given : Footings A1 and A2 of the diaphragm-shearwall concrete office building of Chapter 2, Fig. 2-1, frame line A as shown in the sketch.

Both footings are fixed to the structural elements above.

Reference footing is 14 ft square with depth of founding of 4 ft.

Soil is an overconsolidated clay, CH.



To find: 1) Required sizes of the footings

2) Check the adequacy of the restoring moment

Solution:

Determine the required footing sizes first

Design loads for footings A1 and A2 are summarized in Chapter 3, Example 3-4. The results are:

Design loads for footing A1,

$V = 36$ kips and $M_{ov} = 820$ kip•ft (wind)

$V = 155$ kips and $M_{ov} = 4350$ kip•ft (earthquake)

$P_{DL} = 183$ kips $\quad P_{LL} = 24$ kips

W = 29 kips $\quad E_{ELASTIC} = 155$ kips

Maximum gravity load = 207 kips

Maximum sustained load = 195 kips

Maximum combined load = 223 kips (wind)

Maximum combined load = 317 kips (earthquake)

Design loads for footing A2,

$V = 36$ kips and $M_{ov} = 820$ kip•ft (wind)

$V = 155$ kips and $M_{ov} = 4350$ kip•ft (earthquake)

$P_{DL} = 219$ kips $\quad P_{LL} = 49$ kips

W = 29 kips $\quad E_{ELASTIC} = 155$ kips

Maximum gravity load = 268 kips

Maximum sustained load = 244 kips

Maximum combined load = 278 kips (wind)

Maximum combined load = 372 kips (earthquake)

Allowable pressures on the reference footing are determined in Examples 6-9 and 8-7. The pressures are repeated here for immediate reference.

p_a = 3600 psf for maximum gravity loading,

$P_{ref} = 3600 \times 14 \times 14 = 705$ kips

p_a' = 3240 psf with wind,

$P_{ref} = 3240 \times 14 \times 14 = 635$ kips

p_a' = 2370 psf with earthquake,

$P_{ref} = 2370 \times 14 \times 14 = 465$ kips

p_a'' = 2500 psf to produce a settlement of 1 inch

$P_{ref} = 2500 \times 14 \times 14 = 490$ kips

For fixed column bases, a load magnification of 1.25 is used when allowable pressures are limited by strength, p_a and p_a'.

Selection of footing size for footing A1 on a clay using Equation (6-26):

For maximum gravity load with $P_{ref} = 705$ kips

$$B_1 = \sqrt{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt{\frac{1.25 \times 207000}{705000}} \times 14 = 8.48 \text{ ft.}$$

For maximum combined load with $P_{ref} = 635$ kips

$$B_1 = \sqrt{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt{\frac{1.25 \times 223000}{635000}} \times 14 = 9.28 \text{ ft.}$$

For maximum combined load with P_{ref} = 465 kips

$$B_1 = \sqrt{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt{\frac{1.25 \times 317000}{465000}} \times 14 = 12.9 \text{ ft.}$$

For maximum sustained load with P_{ref} = 490 kips using Equation (8-6):

$$B_1 = \frac{P_1}{P_{ref}} B_{ref} = \frac{195000}{490000} \times 14 = 5.57 \text{ ft., settlement } = 1 \text{ in.}$$

<u>USE SQUARE FOOTING, B = 13 ft 0 in. for footing A1</u>

Selection of footing size for footing A2 on a clay using Equation(6-26):
For maximum gravity load with P_{ref} = 705 kips,

$$B_1 = \sqrt{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt{\frac{1.25 \times 268000}{705000}} \times 14 = 9.65 \text{ ft.}$$

For maximum combined load with P_{ref} = 635 kips,

$$B_1 = \sqrt{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt{\frac{1.25 \times 278000}{635000}} \times 14 = 10.36 \text{ ft.}$$

For maximum combined load with P_{ref} = 465 kips,

$$B_1 = \sqrt{\frac{P_1}{P_{ref}}} B_{ref} = \sqrt{\frac{1.25 \times 372000}{465000}} \times 14 = 14.0 \text{ ft.}$$

For maximum sustained load with P_{ref} = 490 kips using Equation (8-6):

$$B_1 = \frac{P_1}{P_{ref}} B_{ref} = \frac{244000}{490000} \times 14 = 6.97 \text{ ft., settlement} = 1 \text{ in.}$$

<u>USE SQUARE FOOTING, B = 14 ft 0 in. for footing A2</u>

The adequacy of the restoring moment is checked

The restoring moment M_R developed by either footing A1 or A2 is computed as $M_R = P_w {}^{s}/_{2}$, where s is the spacing between the footings. Only the least load (the total dead load) is used to determine the least value of restoring moment; it does not matter that the overturning moment was determined using combined loads.

$$M_R = P_w \frac{s}{2} = (183 + 219)\frac{28}{2} = 5630 \text{ kip} \bullet \text{ft (one panel)}$$

For wind loads, this minimum capacity of the restoring moment must exceed the overturning moment 1640 kip•ft on two panels by 50%. Obviously, it does.

For earthquake loads, this minimum capacity of 5630 kip•ft for the restoring moment must be greater than the overturning moment of 8700 kip•ft on two panels, or 4350 kip•ft on one panel. Obviously it does, so the minimum restoring capacity of the panel foundation is found to be adequate for the imposed load.

The frictional shear resistance is checked

For either footing A1 or A2:

For wind, base shear = 36 kips per panel
For earthquake, base shear = 155 kips/panel

Resisting capacity = μ(DL) = 0.3(183 + 219) = 121 kips < 155 kips (NG)

Conclusion:

The shear panel is found to be stable for wind but not for earthquake. Some reconfiguration of the structure will have to be done in order to reduce the lateral load on these footings. Extending a tie beam to footing A3 to add the frictional resistance of footing A3 is a possible remedy. Whatever remedy is chosen, it is assumed that the base shear on the footings of the shear panel will be reduced to 121 kips or less in the design of the superstructure.

Example 9-4 Continuation of Example 9-3

Given : Interior footing B3 of the diaphragm-shearwall concrete office building of Chapter 2, Fig. 2-1, frame line B. Footing B3 is rigidly fixed to column B3 above.

Interior footings in a braced frame are not subject to combined loads.

Reference footing for calculation of p_a in Example 6-10 is 10 ft square, with depth of founding 4 ft

Reference footing for calculation of p_a'' in Example 8-7 is 14 ft square, with depth of founding 4 ft

Soil is an overconsolidated clay, CH.

To find: Required size of the footing

Solution:

Loads on footing B3 are determined in Examples 2-1 and 2-2:

DL = 214 kips, LL = 114 kips

Since footing B3 is an interior footing in a braced frame, it is not subject to wind or earthquake loads.

For gravity load combinations,

Max. gravity load = DL + LL = 214 + 114 = 328 kips
Max. sustained load = DL + 50%LL = 214 + 57 = 271 kips

Allowable pressures are determined in Examples 6-10 and 8-7, repeated below for immediate reference. The loads on the reference footing are computed from these allowable pressures.

For footing B3,

p_a = 3700 psf for the reference footing 10 ft square

P_{ref} = 3700 x 10 x 10 = 370 kips

p_a' is not applicable

p_a'' = 2500 psf for the reference footing 14 ft square

P_{ref} = 2500 x 14 x 14 = 490 kips

For fixed column bases, a load magnification of 1.25 is used when allowable pressures are limited by strength, p_a or p_a'.

Selection of footing size for footing B3 on a clay:

For maximum gravity load, with P_{ref} = 287 kips and B_{ref} = 10 ft,

$$B_1 = \sqrt{\frac{P_1}{P_{ref}}}B_{ref} = \sqrt{\frac{1.25 \times 328000}{370000}} \times 10 = 10.5 \text{ ft.}$$

For maximum sustained load with P_{ref} = 490 kips and B_{ref} = 14 ft,

$$B_1 = \frac{P_1}{P_{ref}}B_{ref} = \frac{271000}{490000} \times 14 = 7.74 \text{ ft., settlement} = 1 \text{ in.}$$

USE SQUARE FOOTING, B = 10 ft 6 in.

Presumptive Bearing Pressures

Long before the advent of soil mechanics, foundations were designed successfully in many areas using soil pressures and foundation types proven by years of experience to be satisfactory for that particular locale. The fact that theoretical soil mechanics has since evolved does not invalidate that earlier experience. In many of the older urban areas of the world, the traditional types of foundations and the allowable soil pressures for that area are quite valid today. Their continued use is certainly justified.

In other areas, however, such experience (and records) simply do not exist. Bustling urban developments exist today where there was only a prairie or a swamp a few years ago. In such a locale, there is no experience to draw on when choosing a foundation type or an allowable bearing pressure.

Often, however, there is a strong tendency to project the traditional proven methods into a new locale, bypassing the more staid practices of conventional foundation engineering. One such practice is the use of presumptive bearing pressures. Presumptive bearing pressures are allowable soil pressures found to be acceptable in one geographic area and presumed to be acceptable for all other areas. A brief table of such presumptive pressures is presented in Table 9-3. Sowers[36] lists a much wider range of presumptive values, developed over his many years in the practice.

It should be noted that the values given in Table 9-3 are subject to the correction factors for footing shape. They are also subject to loss of available strength due to lateral loads. Exactly what one should do about submergence of sands is rarely if ever stipulated.

Table 9-3 Presumptive Bearing Pressures for Vertical Loads on Strip Footings

Material	Allowable Bearing (ksf)
Sand	
Loose	1 to 3
Medium	3 to 6
Dense	6 to 12
Clay	
Soft	0.5 to 1.5
Firm	1.5 to 2.5
Stiff	2.5 to 6.0
Rock	
Fractured	10 to 20
Seamed	20 to 50
Massive	50 to 200

Presumptive values are often prescribed by the building code for a city. One such building code used throughout much of the western United States is the Uniform Building Code[21], which contains presumptive values not only for bearing pressure but also for lateral earth pressures. Unfortunately, the code is often adopted by a city without being amended to suit local limitations and conditions. A foundation code that applies to the expansive clays of Oklahoma is not likely to apply in the same way to the low-plasticity clays of California. In addition, it is rarely clear whether presumptive values are governed by strength or by settlement. The designer must make a judgement (or a guess) in such matters.

Even though the use of presumptive soil pressures is frowned upon by soils engineers, they are in fact widely used, particularly for preliminary design. In most cases, presumptive values are overly conservative (but not always) and produce a foundation that costs more than it should. On small projects, however, the additional cost is usually far less than what it would cost to perform a soils investigation.

The engineer who uses presumptive values in a design remains professionally responsible and personally liable for the adequacy of the design.

Variation of Contact Pressures under a Footing

In earlier discussions, it was tacitly implied that the contact pressure between a footing and the supporting soil is uniformly distributed. For short-term loading, this implication is not accurate. For long-term loading, it may come somewhat closer to being true.

The contact pressure under an absolutely rigid footing is shown in Fig. 9-10. The pressure distribution is shown in Fig. 9-10a for sands and in Fig. 9-10b for clays[13]. As might be expected, the responses of the two soils to this load condition are diametrically opposite. It is, however, a localized effect, with the average pressure being the same for both cases.

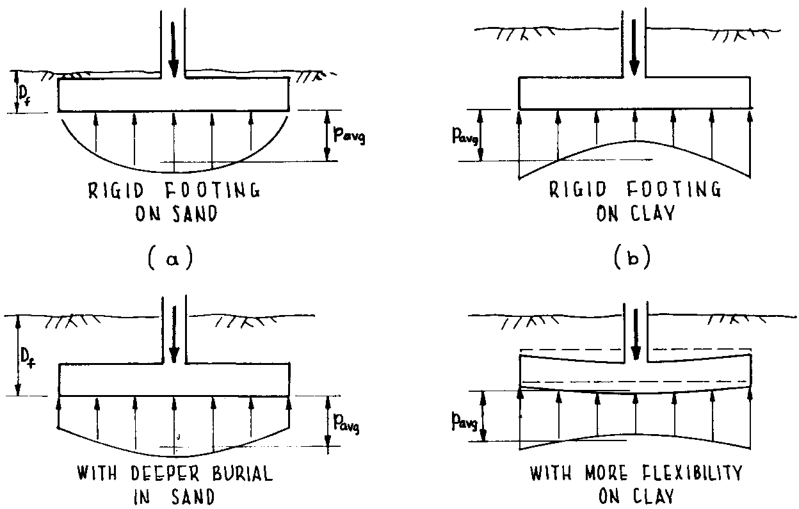


Figure 9-10 Contact Pressures under a Rigid Footing

The confining pressure under the edges of a rigid footing on sand will diminish somewhat toward the edges of the footing, allowing the sand at the edges of the footing to move laterally. The effect is to relieve slightly the pressure at the edge of the footing. The end result is the pressure distribution shown in Fig. 9-10a, where the sand confined near the center of the footing will eventually take more of the load. Placing the footing at a lower depth, thereby increasing D_f and the confining pressure, will help to produce a more uniform distribution.

No footing is absolutely rigid, however. As the spread footing of Fig. 9-10a is loaded, the edges deflect upward, relieving further the pressure at the edges of the footing. The result is to aggravate the nonuniform pressure distribution. The effects of the flexural deflections of the footing are not relieved by placing the footing at a lower elevation; they occur whatever the founding depth D_f may be.

The nonuniform distribution is relieved slightly with time, as the sand at the higher pressures undergoes more fragmentation, more creep and more settlement than the sand at the lower pressures. As the settlement thus increases at the center, the edges then pick up more load, thereby equalizing the pressures to some degree. It is doubtful, however, whether the pressure distribution ever approaches a uniform distribution.

In sands, the increase in the vertical pressure at the center of a footing is not an insignificant amount. It can be as much as 40% above the average pressure[13] and can increase the lateral pressures under the center of the footing by as much as 20%. Superficially, the size of these increases would seem to justify including these effects in the design calculations.

It must be remembered, however, that the strength of a sand increases with the confining pressure. As the vertical pressure increases, the confining pressure increases; strength increases correspondingly. As a result, the strength of the sand toward the center of the footing is actually stronger than the sand at the edge of the footing.

When the foregoing effects of the increased fragmentation, increased creep and increased strength are considered together, the overall effect of nonuniform contact pressures in sands become much less serious. In fact, it is rarely considered for footings of the more ordinary sizes in routine structures. The effects can become notable, however, for larger footings (B > 10 ft) and for mat and raft foundations. Advanced textbooks[12] treat the subject in detail.

In clay soils, the pressure variation is opposite to that of sands. In clays, the unequal deformations between the loaded clay particles and the adjacent unloaded clay particles alog the edge of the rigid footing serve to increase sharply the pressures at the edges of the footing. The unloaded clay particle just outside the loaded area go into tension, pulling upward on the adjacent loaded particles. The result is the sharp nonuniform pressure gradient shown in Fig. 9-10b. Placing the founding line at a higher or lower elevation will not relieve the condition, since contact pressures in clays are relatively unaffected by confining pressures.

In contrast to sands, however, the nonuniform pressure distribution in clays are relieved considerably by the upward flexural deflections that must occur at the edges of the footing. Additionally, with the higher pressures that exist at the edges of the footing, the clay must undergo a higher degree of consolidation and creep at the edges of the footing, thereby relieving the pressure concentrations even further. With time, the pressure distribution will become much closer to a uniform distribution.

As with sands, the effects of nonuniform distribution of pressures in foundations on clay is rarely a design consideration. Such effects are not included in this book. The problem does become more serious in larger foundations, however, to the extent that it becomes a routine design consideration in large-scale mat and raft foundations[12,14].

Review Questions

9.1 In soils classified as clays, the contribution of the friction component to the strength of the soil (if any) is usually ignored. Why?

9.2 In soils classified as sands, the contribution of the cohesion component to the srength of the soil (if any) is usually ignored. Why?

9.3 Insofar as bearing pressures are concerned, what is the advantage in hinging a footing to the structural element that it supports?

9.4 Why aren't all footings hinged to the structural elements that they support?

9.5 How can moment at the top of a column cause rotations of a footing at the bottom of the column?

9.6 What is the difference between settlement and differential settlement?

9.7 What is the generally accepted value for allowable settlement of a footing? At this level of total settlement what will likely be the maximum differential settlement between adjacent footings?

9.8 How much settlement is generally assumed when the allowable bearing strength of the soil is being determined?

9.9 In a spread footings, where does the average settlement occur?

9.10 What is the effect of column moments on average settlement?

9.11 In a typical soil at a shallow depth of founding, how much resistance to rotation can be developed by a spread footing?

9.12 At a typical spread footing supporting a column, how much fixity of the column base can be developed?

9.13 About how much increase in footing pressure can be expected due to column moments from any source? Why the limitation?

9.14 What feature must be used in connecting a column to its footing if significant footing rotations are expected to develop?

9.15 How can one provide columns fixed at their bases (their foundation) when such fixity becomes necessary?

9.16 At a typical spread footing supporting a concrete column in a rigid frame, where does the point of rotation (the hinge) at the base of the column actually occur?

9.17 When a column is rigidly fixed to its isolated spread footing and the strength criteria is the limitation under consideration (p_a or p_a'), how is the peak in soil pressure incorporated into the design?

9.18 When a column is rigidly fixed to its isolated spread footing and settlement criteria is the limitation under consideration (p_a"), how is the peak in soil pressure incorporated into the design?

9.19 Where does one usually find the values for presumptive bearing pressures that have been legally adopted for a particular locale?

9.20 In the event of a foundation failure due to the use of legally adopted presumptive bearing pressures, who is liable?

9.21 In a sandy soil, is the contact pressure at the edge of a spread footing higher or lower than that at the center of the footing? In a clay soil?

9.22 In a sandy soil, what causes the pressure under a spread footing to become more nearly uniform with time? In a clay soil?

9.23 In a sandy soil, does making the footing more flexible relieve or worsen the nonuniform pressure distribution? In a clay soil?

9.24 In a clay soil, how can one estimate the allowable bearing pressure that will produce 1 inch settlement, with no tedious theoretical calculations or expensive consolidation tests?

9.25 In a reasonably well-compacted sandy soil, how can one estimate the allowable bearing pressure that will produce 1 inch settlement, with no tedious theoretical calculations or expensive Dutch cone tests?

OUTSIDE PROBLEMS

9-1 About how much contact pressure under a square spread footing will produce a settlement in the order of 1 inch in a medium clay? γ= 102 pcf, D_f= 3'-0", c = 1200 psf

9-2 About how much contact pressure under a square spread footing will produce a settlement in the order of 1 inch in a loose sand?

9-3 A spread footing 8 feet square carries a load of 256 kips, exerting a bearing pressure of 4000 psf on the supporting soil. The footing is forced to undergo a rotation in its vertical plane of 0.20°. Determine the changes in soil pressure that occur due to this rotation, assuming that a soil pressure of 4000 psf corresponds to a pressure of 1 inch.

9-4 A spread footing 8 feet square is is used to support a concrete column 14 in. square and 14 ft. long to the girder at its upper end. Assume that the column is effectively fixed against rotation by the girder at its upper end. Modulus of elasticity E_c of the concrete is 3,600,000 psi, ultimate stress f_c' is 4000 psi and allowable stress f_c is 1800 psi.

The footing undergoes a rotation of 0.24° due to soil conditions. Use the equations given with Fig. 9-3 to find the amount of stress that is induced back into the column.

9-5 through 9-24 Determine the required contact area for the spread footings or strip footings under the tabulated load and attachment conditions. Pressure p_a'' is the pressure at 1 in. settlement of the reference footing.

		Allowable pressures			External loads								
					Spread Footings			Strip Footings					
Prob.	Ftg. type	pa ksf	pa' ksf	pa" ksf	DL kips	LL kips	W or E kips	DL klf	LL klf	W or E klf	Attach-ment	Soil	Ref. Ftg ft.
9-5	Square	4.0	3.2	3.0	100	75	21				Hinged	SM	8x8
9-6	Strip	4.0	3.2	3.0				9	7	3	Hinged	SC	4
9-7	Square	2.6	2.1	2.0	100	100	11				Fixed	SP	9x9
9-8	Strip	2.6	2.1	2.0				7	9	5	Fixed	SW	6
9-9	Square	3.2	2.5	2.5	75	60	9				Fixed	ML	5x5
9-10	Strip	3.2	3.2	3.0				8	8	0	Fixed	CL	5
9-11	Square	2.8	1.9	2.0	75	75	9				Hinged	MH	8x8
9-12	Strip	2.6	2.1	2.0				8	8	0	Hinged	CH	7
9-13	Square	2.2	1.9	1.8	90	80	0				Fixed	SM	8x8
9-14	Strip	2.2	1.9	1.8				7	9	4	Hinged	ML	8
9-15	Square	2.2	1.9	1.8	70	100	0				Hinged	SC	8x8
9-16	Strip	2.2	1.9	1.8				9	7	4	Fixed	CL	7
9-17	Square	2.6	2.1	2.0	75	100	18				Hinged	ML	7x7
9-18	Strip	2.8	1.9	2.0				10	6	0	Hinged	ML	7
9-19	Square	3.2	3.2	3.0	80	120	21				Fixed	SM	9x9
9-20	Strip	3.2	2.5	2.8				12	8	0	Fixed	SM	6
9-21	Square	2.6	2.1	2.0	100	120	24				Fixed	SM	9x9
9-22	Strip	2.6	2.1	2.0				14	9	6	Fixed	SM	8
9-23	Square	4.0	3.2	3.2	150	125	31				Hinged	ML	7x7
9-24	Strip	4.0	3.2	3.0				16	9	7	Hinged	ML	7

COMPREHENSIVE PROBLEMS

(Recommended for use as one-week outside problems or as part of a take-home hour examination)

9-25 A rigid frame structure is 168 feet square in plan, having column modules of 28 feet in both directions. Depth of founding of all footings is 4 feet.

An interior column in an interior frame line of the structure carries a dead load of 94 kips and a live load of 116 kips. Total base shear along the frame line is 90 kips due to wind. Center of lateral loads h_{LAT} is 26.2 feet above top of footings. The column is hinged to its square spread footing.

The supporting soil is loose sand with a unit weight of 119 pcf and a field SPT blow count of 9 at a depth of 9 feet. The site is not subject to groundwater intrusion.

Select a size for a reference footing and determine the allowable bearing pressure for strength limitations. Factor of safety to bearing failure is 2.5. The estimated values given by Table 9-3 for p_a'' are to be used for settlements.

Determine the required size of this interior footing.

9-26 For the rigid frame of Problem 9-25, select the required size of the perimeter footing at the at the end interior frame line.

9-27 A braced frame structure 120 feet x 240 feet in plan has a column module of 24 feet in both directions. An interior column in the structure carries a dead load of 169 kips and a live load of 140 kips. All footings are founded at 4 feet deep and are fixed to their structural elements above.

The supporting soil is a clay with an unconfined compressive strength of 2200 psf and a unit weight of 111 pcf.

Select a size for a reference footing and determine the allowable bearing pressure for strength limitations. Factor of safety to bearing failure is 2.5. The estimated values given by Table 9-3 for p_a'' are to be used for settlements.

Determine the required size of this interior footing.

9-28 For the braced frame of Problem 9-27, select the required size for a shearwall footing for the following column loads:

At one end of the panel,	DL = 112 kips,	LL = 99 kips
At the other end,	DL = 196 kips,	LL = 174 kips
Total shear on the panel	V = 124 kips due to wind	

Chapter 10

COMPARATIVE SELECTION OF FOOTING SIZES*

Interaction within a Group of Footings

Footings rarely occur singly. Foundations are composed of groups of footings, sometimes with footings in close proximity to each other, sometimes with large footings adjacent to small footings, sometimes with spread footings adjacent to strip footings. In addition to the direct interaction between a single footing and its supporting soil, there can also be a secondary or induced interaction between any one footing and the footings adjacent to it.

Not only is there a possible interaction from footing to footing, there may also be an interaction from one soil stratum to an underlying weaker (or more compressible) soil stratum. This stratum-to-stratum interaction can obviously contribute directly to settlements. It can also induce further interactions or secondary effects from footing to footing.

The propagation of effects from footing to footing or from stratum to stratum occurs due to the lateral spread of the Boussinesq pressure bulb. While dispersion of pressure occurs rapidly and vertical pressures decrease sharply in only a few feet, the pressure can still cause undesirable effects laterally. This chapter is devoted to a brief study of this dispersion of pressure under a footing both vertically and laterally and the effects that such a dispersion can have on neighboring footings.

A key property of shallow foundations is that large loads with their large footings produce larger pressure bulbs than smaller loads with their smaller footings. At the same contact pressure, large footings with their large pressure bulbs will therefore compress a larger volume of soil. Consequently, the larger volume of soil will undergo more vertical settlement than a smaller volume will undergo.

Some notable conclusions may be drawn from the foregoing observations which will be useful in later comparisons. In a group of footings having relatively comparable contact pressures,

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* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

- The largest footing will settle most, or equivalently,
- The footing carrying the largest load will settle most, or as a corollary,
- The footing carrying the smallest load will settle least

It is again noted that in the approach used in this book, the actual magnitude of settlements at a particular footing is rarely of concern. The overriding concern is the difference between the settlements of one footing as compared to the settlements of other footings in its vicinity. This comparison of relative settlements rather than prediction of absolute values is utilized repeatedly in the following discussions.

Relative Settlements Between Footings

As stated at the beginning of Chapter 8, the maximum estimated settlement of the larger footings will usually be limited to 1 in. The difference in settlements between any two adjacent footings, large or small, will then be somewhat less than 1 in., presumably not more than $^3/_4$ in. In general, properties of soils are neither dependable enough nor consistent enough to justify any higher level of confidence than this. Thus it is not known exactly what the settlement of any footing will be at any one time, only that the differential settlements overall will be kept within certain bounds.

Such a blanket approach is easy to defend, in view of the hundreds of load conditions that might occur on a foundation. The foundations, in turn, must be supported by a soil whose properties can change after each rainstorm. A blanket solution is one of the few practical approaches that can be applied under such a multiplicity of possible conditions.

Even within this approach, however, there has long been a trend in the industry to try to equalize the settlements under all footings as much as possible. Such problems in "proportioning for equal settlements" have even appeared occasionally on licensing examinations for civil engineers. Some federal agencies[16,41] require that buildings built under their auspices include such provisions in the foundation design. A brief discussion of such proportioning is therefore appropriate even in an elementary textbook such as this..

The concept of a nonexistent reference footing of an arbitrarily chosen size was introduced earlier both for strength limitations and for settlement limitations. For the design of a group of footings, the allowable pressures p_a, p_a' and p_a'' are determined for this one reference footing. The sizes and settlements of all other footings in the group can then be determined by comparison to this reference footing.

The relationships developed in earlier chapters are collected for convenience into the following single summary.

- **On sands**, comparison of footing i to a reference footing

  If the two footings are designed at the maximum allowable strength of a sand, the footing loads P will be proportional to the cube of the widths B,

  $$\frac{P_i}{P_{ref}} = \frac{B_i^3}{B_{ref}^3} \quad \text{with settlements} \quad \frac{S_i}{S_{ref}} = \frac{P_i / B_i}{P_{ref} / B_{ref}}$$

  If the two footings are designed for equal settlement in the sand, the footing loads P will be directly proportional to the widths B,

  $$\frac{P_i}{P_{ref}} = \frac{B_i}{B_{ref}} \quad \text{with settlements} \quad \frac{S_i}{S_{ref}} = \frac{P_i / B_i}{P_{ref} / B_{ref}}$$

- **On clays**, comparison of footing i to a reference footing

  If the two footings are designed at the maximum allowable strength of a clay, the footing loads P will be proportional to the square of the widths B

  $$\frac{P_i}{P_{ref}} = \frac{B_i^2}{B_{ref}^2} \quad \text{with settlements} \quad \frac{S_i}{S_{ref}} = \frac{P_i / B_i}{P_{ref} / B_{ref}}$$

  If the two footings are designed for equal settlement in the clay, the footing loads P will be directly proportional to the widths B,

  $$\frac{P_i}{P_{ref}} = \frac{B_i}{B_{ref}} \quad \text{with settlements} \quad \frac{S_i}{S_{ref}} = \frac{P_i / B_i}{P_{ref} / B_{ref}}$$

Applications in Selecting Footing Sizes

The following example illustrates the use of the foregoing comparisons. No attempt is made in this first example to produce equal settlements at each footing. The settlement of the reference footing is selected arbitrarily as the reference settlement against which all other settlements are compared.

Example 10-1 Computation of Reference Settlements.

Given : Group of interior footings in a steel braced frame, founded on a deep stratum of sand at a depth of 3 ft.

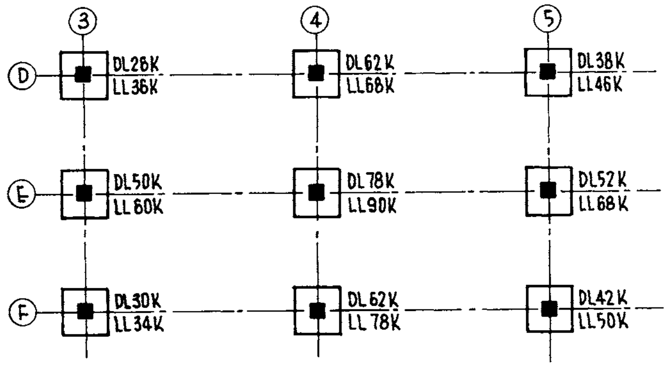
All footings are hinged to their columns.

A footing 6 ft square is arbitrarily chosen as a reference footing size.

For this reference footing, a side evaluation of Equation 6-15 indicates an allowable pressure p_a of 3000 psf at maximum gravity loading, DL + LL.

Gravity loads are shown in the sketch.

To Find: 1) Select proper footing sizes at maximum allowable pressure on the sand.
2) Compare settlements at all other footings to the settlement of the reference footing



Solution:

Footing sizes as limited by strength and the accompanying settlements are computed by comparison:

$$\frac{P_i}{P_{ref}} = \frac{B_i^3}{B_{ref}^3} \quad \text{with settlements} \quad \frac{S_i}{S_{ref}} = \frac{P_i / B_i}{P_{ref} / B_{ref}}$$

In the following tabulation, the required width B of each footing is computed as the cube root of the ratio of loads P_i/P_{ref} times B_{ref} and is then rounded to the nearest 6 inches. The actual pressure is then P_i/B_i^2.

Relative settlements are then computed for the resulting widths B.

Footing	DL kips	LL kips	DL + LL kips	Width B	Pressure ksf	Relative settlement
Ref.			108	6 ft 0 in.	3.00	1.00 S_{ref}
D3	26	36	62	5 ft 0 in.	2.48	0.69 S_{ref}
D4	62	68	130	6 ft 6 in.	3.08	1.11 S_{ref}
D5	38	46	84	5 ft 6 in.	2.78	0.85 S_{ref}
E3	50	60	110	6 ft 0 in.	3.06	1.02 S_{ref}
E4	78	90	168	7 ft 0 in.	3.43	1.33 S_{ref}
E5	52	68	120	6 ft 0 in.	3.33	1.11 S_{ref}
F3	30	34	64	5 ft 0 in.	2.56	0.71 S_{ref}
F4	62	78	140	6 ft 6 in.	3.31	1.20 S_{ref}
F5	42	50	92	5 ft 6 in.	3.04	0.92 S_{ref}

In Example 10-1, some variation of the soil pressure occurs as a result of rounding off the width B to the nearest 6 in. For footings designed at maximum allowable strength, it should be expected that there will be considerable variation in settlements. For example, the least settlement occurs at the smallest footing D3, which settles only 52% as much as the largest footing E4. Even so, if the settlement of footing E4 is known to be less than 1 in., all settlements will be well within the $^3/_4$ in. differential settlement that is usually accepted.

It must be remembered that every footing in the group will have a different allowable pressure, depending on its width B. For the footings of Example 10-1, the maximum allowable bearing pressure is the pressure listed in the table. As indicated, some footings will have an allowable pressure higher than that of the reference footing, some lower.

Alternatively, the footings of Example 10-1 might also be proportioned for equal settlements. The resulting footing widths B for settlement limitations may not be smaller than the required widths B for strength limitations (listed in Example 10-1), but they may be made considerably bigger if desired.

In the next example, the widths of the footings of Example 10-1 are proportioned to produce equal settlements.

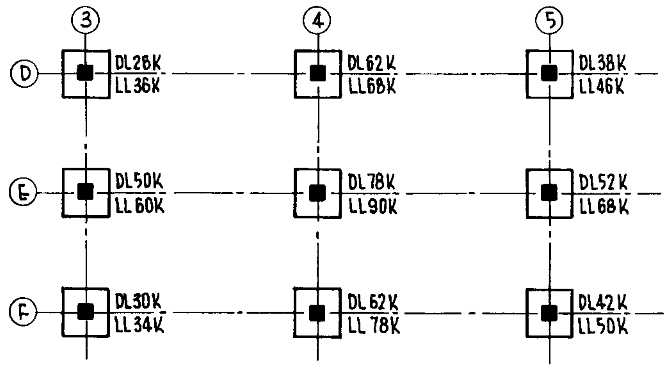
Example 10-2 Proportioning for equal settlements

Given : Group of footings of Example 10-1 The reference footing size of 6 ft square is retained for this calculation.

The Schmertmann solution for the reference footing on this soil was performed in a side calculation, indicating that a settlement of 1 in. will occur at a pressure of 1.9 ksf.

Gravity loads are shown in the sketch.

To Find: Proper footing sizes to produce equal settlements at all footings in the group.



Solution:

Footing sizes and settlements are computed by comparison:

$$\frac{P_i}{P_{ref}} = \frac{B_i^3}{B_{ref}^3} \text{ with settlements } \frac{S_i}{S_{ref}} = 1 = \frac{P_i / B_i}{P_{ref} / B_{ref}}$$

In the following tabulation, the dimension B is computed as the ratio of loads times B_{ref} and is then rounded to the nearest 6 inches. The actual pressure p_a is then P/B^2.

Due to the rounding, the actual settlements will not be exactly equal. To obtain an indication of the final accuracy, the actual settlements have been computed for comparison.

				At Equal Settlement			Final Choice	
	DL	50%LL	DL+50%LL	Width	Press.	Settmt	Width	Settmt
Ftg.	kips	kips	kips	B	ksf	in.	B	in
Ref.			68	6 ft 0 in.	1.90	1.00	ok	1.00
D3	26	18	44	4 ft 0 in.	2.75	0.97	5 ft 0 in	0.78
D4	62	34	96	8 ft 6 in.	1.33	1.00	ok	1.00
D5	38	23	61	5 ft 6 in.	2.02	0.98	ok	0.98
E3	50	30	80	7 ft 0 in.	1.63	1.01	ok	1.01
E4	78	45	123	11 ft 0 in.	1.02	0.99	ok	0.99
E5	52	34	86	7 ft 6 in.	1.53	0.95	ok	0.95
F3	30	17	47	4 ft 0 in.	2.94	1.04	5 ft 0 in	0.83
F4	62	39	101	9 ft 0 in.	1.25	0.99	ok	0.99
F5	42	25	67	6 ft 0 in.	1.86	0.99	ok	0.99

The increase in some of the footing sizes between Examples 10-1 and 10-2 is significant. To produce equal settlements, the larger footing sizes had to be increased markedly. For example, footing E4 was 7 ft 0 in. in Example 10-1 and increased to 11 ft 0 in. in Example 10-2, some 60% increase. Usable pressures were of course reduced accordingly. It can be seen immediately that proportioning for equal settlements does involve additional cost.

The additional cost, however, is primarily that of bulk concrete. Whether the extra cost is warranted can only be decided by the designer, in consideration of the particular structure and the particular site and the particular client at the particular time. Many designers, however, routinely design all foundations for equal settlements.

In Example 10-2, the soil pressure was limited by settlement criteria. For settlements to be kept equal, the settlements of those footings larger than the reference footing had to be reduced to match that of the reference footing; their contact pressures are therefore less than the 1.5 ksf of the reference footing. Conversely, the settlements of those footings smaller than the reference footing had to be increased to match that of the reference footing; their contact pressures are therefore greater than the 1.5 ksf of the reference footing. In no case, however, should the width required to meet settlement limitations be less than the width required to meet strength limitations, as listed in Example 10-1.

Accordingly, the required widths for strength in Example 10-1 are now compared to the required widths for settlements in Example 10-2. The comparison indicates that for two footings, D3 and F3, the width must be 5 ft or more in order to meet strength limitations but may not be more than 4 ft in order to meet settlement limitations. There is, therefore, no footing width for these two footings that will meet both of the conflicting requirements for strength and settlement.

As indicated in the tabulation of Example 10-2, if the final width of D3 is selected to be 5 ft, as required for strength, the settlement of footing D3 will be only 0.78 in., some 22% less than the desired 1 inch. Similarly, the settlement of footing F3 will be only 0.83 in. some 17% less than the desired 1 in. The goal of proportioning all footings in the group to settle equally would therefore not be achieved.

Two possibilities might be considered under such conditions:

1) Select the largest width for all footings, regardless whether the largest width is required for strength requirements or settlement requirements. Recognize and accept that some of the small, lightly loaded footings will not settle far enough to match the settlements of the larger footings; equal settlements will thus not be achieved.
2) Determine which footing will settle least (the one with the smallest load). Determine the settlement of this footing when it is loaded to the maximum allowable strength of the soil (It will be somewhat less than the desired 1 inch). Proportion all other footings sizes such that no footing will settle more than this reduced limiting amount.

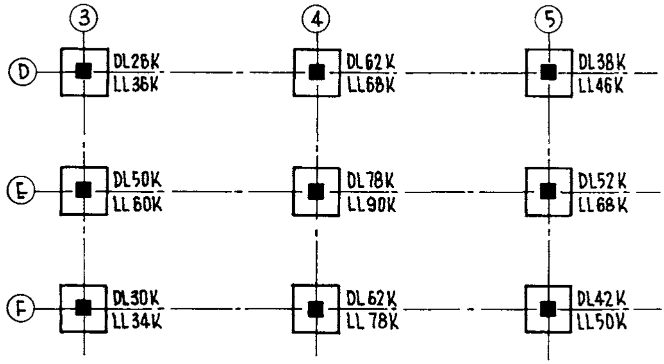
The first possibility is included in Example 10-2. The last two columns of that tabulation indicate the final selection of widths and settlements for the two footings, D3 and F3, that do not meet the goal of equal settlements. In this particular case, the discrepancy is obviously not a serious one.

The second possibility is presented in the following continuation of the analysis.

Example 10-3 Proportioning for equal settlements.

Given : Group of footings of Examples 10-1 and 10-2. Footing D3 is taken as the smallest footing, with a sustained gravity load of 44 kips. Minimum width is 5 ft 0 in. with a corresponding settlement of 0.78 in. as listed in Example 10-2.

To find: Proportions of all footings such that no footing will settle more than the smallest footing.



Solution:

If no settlement can exceed the settlement of 0.78 in. for footing D3, the load on the reference footing must be revised. A new load $P_{ref(new)}$ will be determined for the reference footing. This load will correspond to the limiting settlement of 0.78 in. rather than the 68 kips corresponding to 1 in. settlement that was used in Example 10-2.

$$\frac{S_{ref\,(new)}}{S_{ref\,(old)}} = \frac{P_{ref\,(new)}/B_{ref}}{P_{ref\,(old)}/B_{ref}} \qquad \frac{0.78}{1.00} = \frac{P_{ref\,(new)}/6}{68/6}$$

Hence, $P_{ref\,(new)} = 53$ kips

New footing widths can now be determined for all other footings that will result in equal settlements of 0.78 in. For this calculation, the usual relationship for comparing loads and widths at equal settlements will apply:

$$B_1 = \frac{P_1}{P_{ref\,(new)}} B_{ref}$$

For a check on settlements, the usual relationship for comparing settlements will apply:

$$\frac{S_1}{S_{ref\,(new)}} = \frac{P_1/B_1}{P_{ref\,(new)}/B_{ref}}\ ; \qquad S_1 = 0.78\frac{P_1/B_1}{53/6}$$

The resulting widths and settlements are shown in the following tabulation.

Footing	DL kips	50%LL kips	DL+50%LL kips	Width B	Press. ksf	Settmt in
Ref.			53	6 ft 0 in.	1.47	0.78
D3	26	18	44	5 ft 0 in.	1.76	0.78
D4	62	34	96	11 ft 0 in.	0.79	0.77
D5	38	23	61	7 ft 0 in.	1.24	0.77
E3	50	30	80	9 ft 0 in.	0.99	0.78
E4	78	45	123	14 ft 0 in.	0.63	0.78
E5	52	34	86	10 ft 0 in.	0.86	0.76
F3	30	17	47	5 ft 6 in.	1.55	0.75
F4	62	39	101	11 ft 6 in	0.76	0.78
F5	42	25	67	7 ft 6 in.	1.19	0.79

The widths B found in Example 10-3 can now be compared to the widths B found in Example 10-1. In every case, the width B required to limit settlements to 0.78 in. is equal to or greater than the width B required for strength limitations. The increases in the footing widths are striking, some being as much as doubled. The increases in cost would probably be prohibitive, in consideration of the rather marginal benefits.

The loads and allowable pressures of Examples 10-1, 2 and 3, however improbable, were deliberately contrived in order to produce the conflicting choices that were encountered. Very likely, the final footing widths presented in the tabulation of Example 10-2 would be the final choice, with footing widths D3 and F3 being increased to 5 ft.0 in. as indicated.

Effects of Close Proximity

When two footings are placed close together, their pressure bulbs may overlap, as shown in Fig.10-1. It should be apparent that the pressure at any level in the overlap is simply the sum of the two contributing pressures. The settlements in this zone will of course be commensurately higher than those at the same level at either side.

It is a simple matter to compute the approximate total pressure in the overlap zone. In Fig.10-1 the pressure at the top of stratum 2 is the sum of the two overlapping pressures, 1050 + 700 = 1750 psf. Since the allowable pressure on the lower stratum is only 1200 psf, a serious overstress would seem to occur in this example.

The accuracy of such a calculation is highly questionable. The pressure bulb itself is an approximation, made under the assumption that pressures are constant at any level within the bulb and that the dispersion angle of the bulb is 2 vertical to 1 horizontal. It was shown in Chapter 5, however, that the pressures are not uniform and that the angle is in fact variable. While the average pressure at any level may be a reasonably accurate approximation, using it in computations of a precise pressure at a particular point are not so believable.

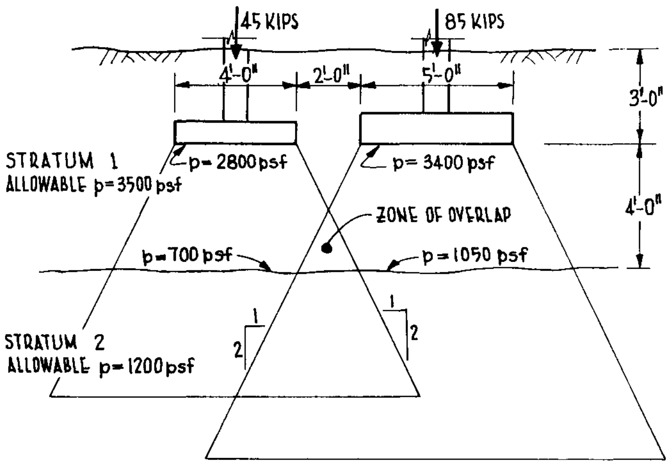


Figure 10-1 Zone of overlap.

When pressure bulbs overlap as shown in Fig.10-1, the settlements can produce a “tilt” of the individual footings toward each other. An exaggerated case of overlap and tilt is shown in Fig.10-2. The exact amount of tilt would, of course, depend on the magnitude of pressures and the compressibility of the soil.

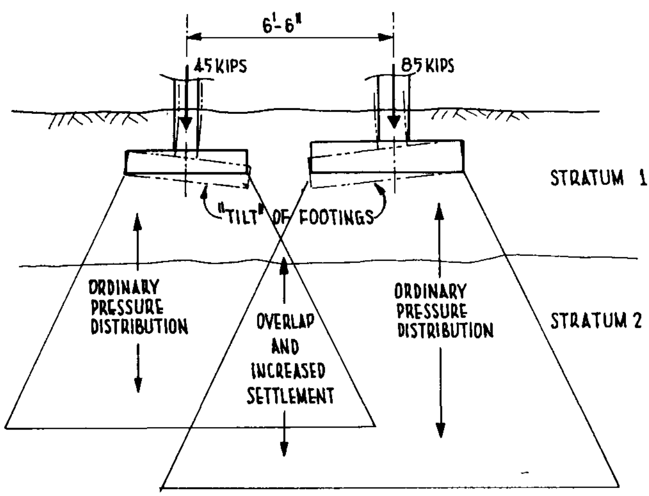


Figure 10-2 Rotations due to overlap.

It was shown in Chapter 9 that even large moments on a rigid elastic column do not produce enough rotations at the footings to cause significant variations in soil pressure. That characteristic, however, is a two-edged sword. The converse conclusion must also be drawn: *when a column is fixed to its footing, even small rotations of the footing can induce significant moments back into the rigid elastic column.* Even the small tilt of the footing shown in Fig.10-2 could therefore easily induce destructive moments back into the column.

As noted earlier, it may be possible to make calculations and come up with some numbers to represent the rotation of the footings of Fig.10-2. Such numbers are rarely believable. It is far more prudent to eliminate the problem than to be forced to rely on such computations.

One way to eliminate the problem would be to combine the two footings of Fig.10-2 into a single footing, thereby eliminating the problem of rotations of the individual footings. Such a combined footing is shown in Fig.10-3.

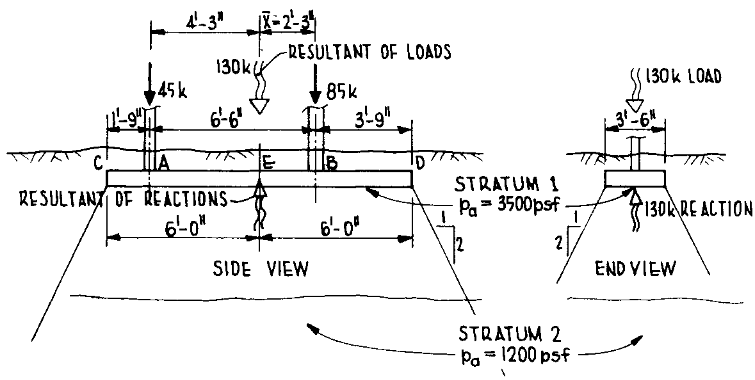


Figure 10-3 Combined footing.

The selection of the dimensions of the combined footing shown in Fig. 10-3 is somewhat arbitrary but entirely straightforward. The dimensions must be set to produce a uniform soil pressure under the combined footing. A uniform soil pressure will be produced if the resultant of loads passes through the centroid of the contact area as shown.

The magnitude of the resultant of the two loads is obviously the sum of the two forces, $R = 45 + 85 = 130$ kips. The location of the resultant is found by simple statics. Moments are summed about point B to find the distance $\bar{x}$,

$$\Sigma M_B = 0 = 45 \times 6.5 - 130\bar{x}; \qquad \bar{x} = 2.25 \text{ ft.} = 2 \text{ ft.} 3 \text{ in.}$$

The distance $\bar{x}$ = 2 ft. 3 in. is shown in Fig. 10-3 above.

The point E shown in Fig.10-3 is thus the center of the rectangular combined footing. It remains only to establish the distances *CA* and *BD* to define completely the required overall length. The distance *CA* is set at an arbitrarily selected 1.75 ft to provide a reasonable amount of clearance around the column at point A, producing the final half-length of 6 ft as shown; the total is then 12 ft.

The width of the combined footing is now set to suit the requirements for contact pressure. For this case, there are two requirements to be met: the contact pressure may not exceed 3500 psf at the founding line; the average pressure in the pressure bulb may not exceed 1200 psf at the top of stratum 2. In the calculations, the width is set to suit the first requirement and is then checked to see that the second requirement is met.

The sum of vertical forces yields a solution for the width w, where the allowable soil pressure is 3500 psf,

$$\sum F_V = 0 = R - pLw = 130{,}000 - 3500 \times 12w$$

$$w = 3.09 \text{ ft} \qquad \text{say, } 3 \text{ ft } 6 \text{ in.}$$

The pressure at the top of stratum 2 is found similarly:

$$\sum F_V = 0 = R - pLw = 130{,}000 - p(12 + 4)(3.5 + 4)$$

$$p = 1083 < 1200 \text{ psf (OK)}$$

The pressure is seen to be less than the allowable pressure of 1200 psf; the dimensions of 3 ft 6 in. x 12 ft for the combined footing are therefore acceptable.

It should be noted that the use of a combined footing in this case has circumvented any need for sophisticated calculations. The pressure under the combined footing is uniform and the danger of erratic settlements has been eliminated. Where column spacing is close, the use of a combined footing sometimes offers an easy and economical means to avoid footing-to-footing interactions.

Effects of Unequal Loads

When two adjacent footings carry loads having highly different magnitudes, the larger load may cause undesirable settlements far below the adjacent smaller load. Such a case is shown in Fig. 10-4. Although the interaction may be more severe where footings are spaced closely together, the effects can also occur to some degree even at normal column spacings of 20 ft or so.

The occurrence of a very lightly loaded column being placed adjacent to a very heavily loaded column is not a rare feature in building design. A covered colonnade adjacent to a multistory building produces such a circumstance, as does a covered loading area adjacent to an industrial building. Wherever such a feature occurs, it should be recognized in the foundation design as a special condition.

It should be obvious from Fig.10-4 that the existence of the small footing will have little or no effect on the large footing. The small pressure bulb may or may not intersect the large one, but either way, its pressures are so small compared to the pressures in the large bulb that its effects on the large bulb can be ignored.

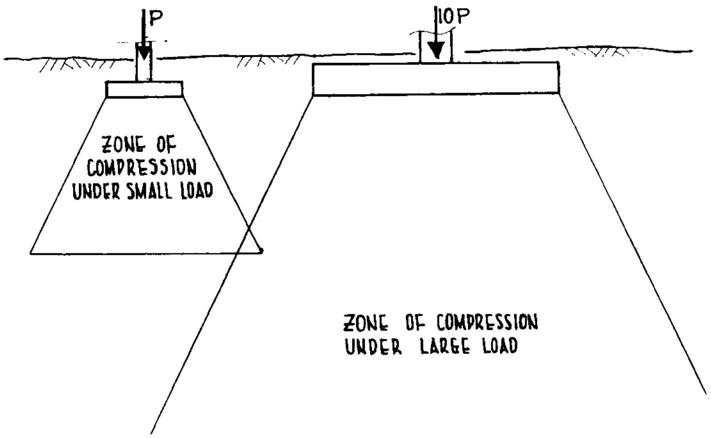


Figure 10-4 Influence of unequal loads.

The pressure bulb of the large footing, however, can extend entirely underneath the small footing as indicated in Fig.10-4. Any settlements produced by the large bulb will produce some degree of settlement in the small footing above. Depending on relative sizes and locations, the small footing will be displaced downward and might also be rotated somewhat.

Since the maximum settlement of the large footing is itself limited to 1 in., any settlements it may induce into the small footing will certainly be less than that. The vertical displacements by themselves will therefore be within the usual design limitations and need not be considered further. The potential rotations of the small footing, however, merit some attention.

In the preceding section, it was found that when two footings are in very close proximity, a combined footing can eliminate the problem of induced rotations. A combined footing might also be used here to eliminate the problem of the rotations induced into the small footing. In most cases, however, the problem is not severe enough to warrant such an elaborate solution.

Since the major concern is the rotations of the small footings, a simple solution is to provide an effective hinge between the column and the footing such as one of those shown earlier in Chapter 9. The footing can then rotate without inducing destructive moments back into the column. Where the number of such columns is small, the aditional cost of deliberately hinging the column bases will probably be much less than that of building a combined footing.

The problems incurred in placing a very small column load near a very large column load seem to receive a lot of attention in the literature. Either of the two possible solutions to the problem suggested above, that is, either using a combined footing or hinging the smaller column to its footing will circumvent the problem. Solutions more sophisticated than these do not seem warranted.

Effects of Intermixed Footing Types

At equal contact pressures and footing widths, strip footings will settle considerably more than spread footings; the difference is estimated herein to be roughly twice as much. The disparity occurs since the pressure under a spread footing is dispersed in two directions; its pressure bulb therefore diminishes quite rapidly. In contrast, the pressure under a strip footing can disperse in only one direction; pressures therefore disperse more slowly and the pressure bulb goes deeper. A sketch showing the two pressure bulbs is shown in Fig. 10-5.

The disparity does not produce as serious a problem as it may appear at first glance. The comparative settlement analyses of Chapter 8 permit a rough comparison of the two settlements:

$$\frac{S_{SPREAD}}{S_{STRIP}} = \frac{p_{SPREAD}B_{SPREAD}}{2p_{STRIP}B_{STRIP}} \tag{8-8}$$

where S_{SPREAD}, S_{STRIP} = settlements of the two footings
p_{SPREAD}, p_{STRIP} = contact pressures of the two footings
B_{SPREAD}, B_{STRIP} = least widths of the two footings

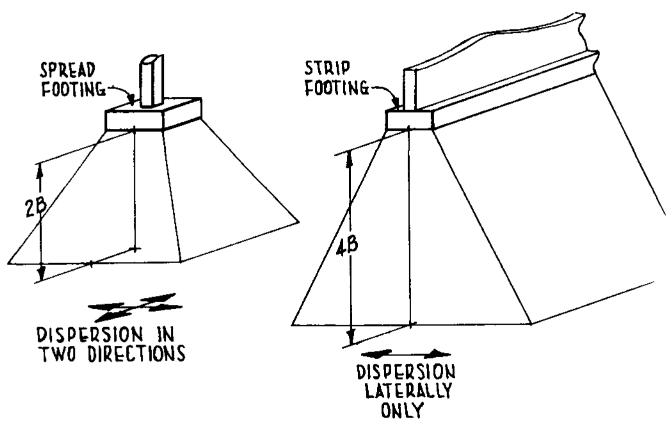


Figure 10-5 Contrasting pressure bulbs.

In terms of loads rather than pressures,

$$B_{SPREAD} = \frac{P_{SPREAD}(lbs)}{2P_{STRIP}(lbs\ /\ ft)} \tag{8-9}$$

It was deduced in Section 8-4 that in a mixed group of spread and strip footings, the strip footing supporting the largest load would usually be selected as the reference footing for settlement calculations. The settlement of all other footings in the group would then be adjusted to match the settlement of this largest strip footing. An example will illustrate such calculations.

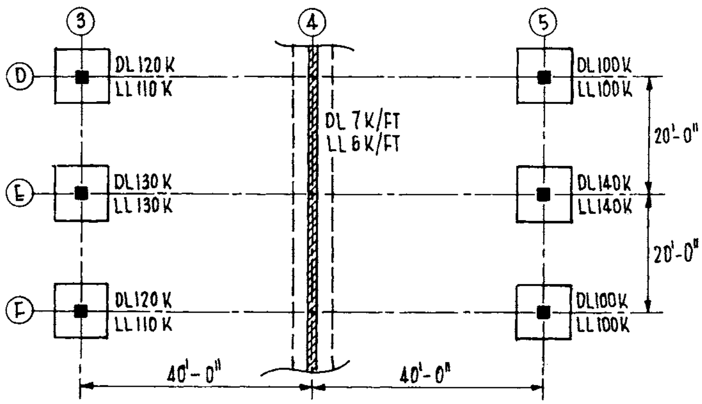
Example 10-4 Comparison of Settlements.

Given : Strip footing intermixed with spread footings at the interior footings of a braced frame, founded on an overconsolidated clay.

Vertical design loads as shown.

For a sustained load of 10,000 lb/ft on a strip footing, a settlement analysis similar to that presented in Example 8-10 indicates that a strip footing 5 ft 6 in. wide will settle roughly 1 inch on this soil under a sustained contact pressure of 1800 psf, or P_{STRIP} = 10 k/ft

To Find: Footing sizes for spread footings that will produce equal settlements



Solution:

Footing sizes are found from Equation (8-9), with the strip footing at line 4 as the reference footing.

Calculations are shown in the following tabulation.

				Strip Footing			Spread Footings		
Ftg	DL k/ft	LL k/ft	DL+50%LL k/ft	DL kips	LL kips	DL+50%LL kips	B ft	Press. ksf	Settmt in.
D3				120	110	175	9' 0"	2.16	0.97
E3				130	130	195	10' 0"	1.95	0.98
F3				120	110	175	9' 0"	2.16	0.97
Line 4	7.0	6.0	10.0				5' 6"	1.82	1.00
D5				100	100	150	7' 6"	2.67	1.00
E5				140	140	210	10' 6"	1.90	1.00
F5				100	100	150	7' 6"	2.67	1.00

In the tabulation, a calculation of the final pressures is included to show the variation in allowable pressure with footing size.

Also included in the tabulation is a check on the settlements to be expected at the given rounded-off footing sizes.

Size and settlement calculations for footing D3 are shown here as an example calculation:

At equal settlements,

$$B_{D3} = \frac{P_{D3}\text{ kips}}{2P_{STRIP}\text{ kips / ft}} = \frac{175}{2\times10} = 8.75\text{ ft.}$$

Pressure after rounding,

$$p''_{D3} = \frac{P_{D3}}{B_{D3}^2} = \frac{175}{9.0^2} = 2.16\text{ kips / ft}^2.$$

Settlement after rounding,

$$S_{D3} = S_{STRIP}\frac{P_{D3}/B_{D3}}{2P_{STRIP}} = 1\times\frac{175/9}{2\times10} = 0.97\text{ in.}$$

The foregoing discussions involved strip footings. It should be apparent, however, that the supporting soil cannot distinguish whether the structure above is a bearing wall on a strip footing or a series of columns on a grade beam. The settlement characteristics of grade beams are obviously identical to those of strip footings.

With this observation that grade beams and strip footings will have the same settlement characteristics, a second alternative design becomes immediately possible. The lines of columns could be supported on grade beams and the bearing wall on a strip footing. Both footings would then be classified as "like" footings and the differences in footing types of Example 10-4 would then be eliminated. The comparison of settlements in this case would revert back to Equation (8-8) and (8-9).

For comparison of settlements between like strip footings, Equation (8-8) becomes:

$$\frac{S_i}{S_{REF}} = \frac{p_i B_i}{p_{REF} B_{REF}} = \frac{P_i\text{ kips / ft.}}{P_{REF}\text{ kips / ft.}} \tag{10-1}$$

At equal settlements, $S_i = S_{REF}$ and

$$P_i = P_{REF} \tag{10-2}$$

Equation (10-2) indicates that for strip footings and grade beams, settlements are a function only of the external line loads along the longitudinal axis of the footing. There is no way to alter the settlement of a strip footing or a grade beam except by altering the external load. Altering the transverse width B will simply increase or decrease the pressure, with no effect on the settlement.

Example 10-4 will now be reexamined using this second alternative design for control of settlements.

Example 10-5 Comparison of Settlements.

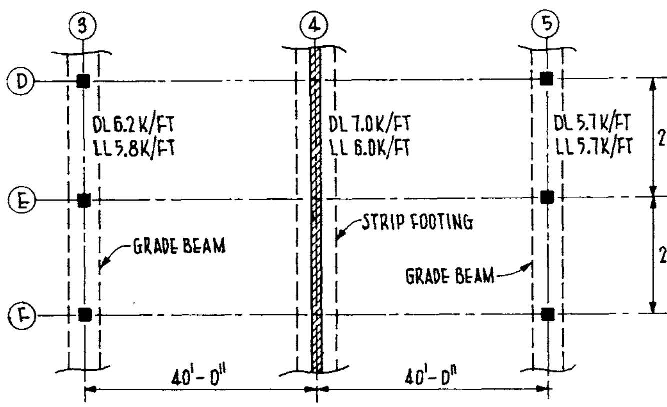
Given : Conditions of Example 10-4 but with columns supported by grade beams rather than spread footings

Vertical design loads as shown.

For a sustained load of 10,000 lb/ft on the strip footing at line 4, a settlement analysis indicates that a strip footing 5 ft 6 in. wide will settle roughly 1 inch on this soil when the sustained contact pressure is 1800 psf (or P_{STRIP} = 10,000 lb/ft)

To Find: 1) Required sizes of the grade beams at lines 3 and 5

2) Comparison of the resulting settlements between the strip footings and the grade beams



Solution:

The uniform line loads along line 3 and line 5 are established by summing the three concentrated dead loads and live loads along the three 20 ft spans and dividing the result by 60. The grade beams would of course be designed to sustain the resulting uniformly distributed load.

An arbitrary choice must be made in order to establish the footing width. If all footings are made 5 ft 6 in wide, the pressure under the grade beam of line 3 will be (6.2 + 2.9)/5.5 = 1.65 psf. Similarly, the pressure under the grade beam of line 5 will be (5.7 + 2.85)/5.5 or 1.55 psf. The grade beams would then be designed as upside-down beams having these uniform contact pressures.

Alternatively, the soil pressures could be made 1800 psf for both the grade beams and the strip footings.

Footing width B_3 would then be (6.2 + 2.9)/1.8 or ft 0 in. and footing width B_5 would be (5.7 + 2.85)/1.8 or 4 ft 9 in.

Since the smaller widths would require less material, it is elected to use this latter alternative:

1) Use $B_3 = 5$ ft 0 in for the grade beam of line 3

   Use $B_5 = 4$ ft 9 in for the grade beam of line 5

The settlement of the grade beams along line 3 are determined by the ratios given in Equation (10-1), where the reference footing is the strip footing at line 4:

$$\frac{S_3}{S_{REF}} = \frac{P_3}{P_{REF}}; \qquad S_3 = \frac{6.2 + 2.9}{7.0 + 3.0} \times 1 = 0.91 \text{ in.}$$

Similarly for the grade beam along line 5,

$$\frac{S_5}{S_{REF}} = \frac{P_5}{P_{REF}}; \qquad S_3 = \frac{5.7 + 2.85}{7.0 + 3.0} \times 1 = 0.86 \text{ in.}$$

2) For a settlement of 1 in at the strip footing of line 4, the settlement of the grade beam at line 3 will be 0.91 in. and the settlement of the grade beam at line 5 will be 0.86 in.

These results indicate that for this case, the settlements are close enough together that they will probably be acceptable.

It is concluded that intermixing spread footings and strip footings does not usually introduce serious problems, even though the different footing types have different settlement characteristics. Even so, the intermixing of footing types should not be taken for granted; where footing types are to be intermixed, a review of their settlements is always warranted.

Effects of Adjacent Excavations

To this point, it has been tacitly assumed that there were no interruptions or voids in the soil in the near vicinity of a footing. Such interruptions do happen, however, and their existence can require that particular attention be paid to the affected footings. As with earlier problem areas, however, it will again be seen that it is far easier to circumvent the problem than it is to deal with it.

A common example of such an interruption is a basement that intercepts the pressure bulb from a nearby footing. A sketch of such a case is shown in Fig. 10-6. The pressure bulb is seen to contact the basement wall well within its zone of influence. The increase in lateral pressure on the basement wall will have a definite effect on the design of the wall. The design of retaining walls, however, is beyond the scope of this book and such a design is not presented here. The concern here is the distortion of the pressure bulb and its effects on the footing.

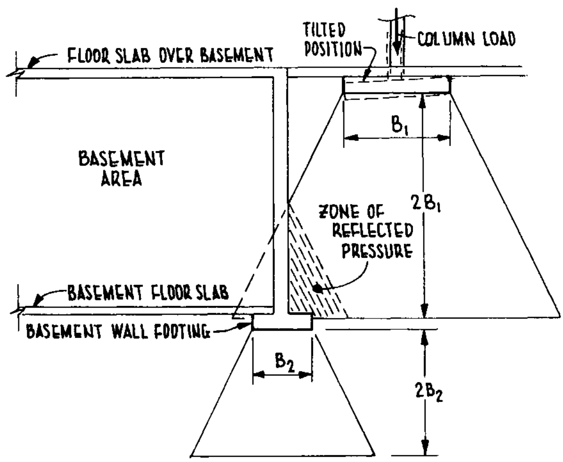


Figure 10-6 Pressure bulb interrupted by basement wall.

From the point where the pressure bulb contacts the basement wall, the pressures are reflected back into the bulb, creating a confused and overlapping pressure buildup in the bulb. The exact pattern of the pressure buildup need not be determined, however. It is enough to recognize that the buildup exists and that it interferes with the normal settlement of the footing.

Since the pressure buildup will occur next to the basement wall, it is presumed that the settlements will increase slightly on that side and the footing will tilt an undetermined amount as shown by the dashed line. The computation of the amount of settlement and the amount of rotation would be very complex calculations. The calculations would depend heavily on having accurate and representative values of the soil properties.

Such calculations are not necessary. It is enough to know that the total settlement will be less than 1 in. and that some rotations may occur. Since settlements are within the allowable amount, they are of no further concern. Any detrimental effects due to rotations of the footing can be avoided simply by assuring that the structural design includes an effective hinge between the column and the footing.

In this case and other cases where discontinuities in the soil stratum produce distortions in the pressure bulb, it may be possible to eliminate the problem in this way. Where settlements are known to be less than 1 in. but where footing rotations pose a potential hazard to the structure above, an effective hinge at the footing may provide an acceptable means to eliminate the hazard.

Review Questions

The following questions are general in nature. In some cases, answers may be found in this chapter. In other cases, answers may be found in earlier chapters. In all cases, the answers are essential to understanding soil-structure interaction in the design of shallow foundations.

10.1 How are footing-to-footing interactions generated?

10.2 What is the approximate side slopes of the approximate pressure bulb insofar as settlement calculations are concerned?

10.3 How deep is the approximate pressure bulb assumed to extend under a square spread footing?

10.4 How deep is the approximate pressure bulb assumed to extend under a strip footing or a grade beam?

10.5 What is the assumed pressure at the bottom of the approximate pressure bulb for a square footing? For a strip footing or grade beam?

10.6 What happens to the theoretical dispersion of pressure when the approximate pressure bulb extends through two or even three strata of different types of soil?

10.7 When pressure bulbs from two adjacent footings overlap, what is the primary effect on the footings themselves?

10.8 When stratified layers of soil are far out of level, what particular hazard might occur in the soil strata under a building?

10.9 In a diaphragm-and-shearwall structure, what problems might be encountered in proportioning for equal settlements?

10.10 How does comparison of settlements rather than computation of absolute settlements provide more believable results?

10.11 When proportioning for equal settlements, how is it possible to increase the allowable soil pressure under small footings by as much as 50% with no adverse effects?

10.12 Why is a slight "tilt" of a footing due to interacting pressure bulbs such a potential hazard to the structure above?

10.13 What is the usual remedy to offset "tilt" in a spread footing?

10.14 Why are calculations concerning footing rotations or tilting so subject to doubt?

10.15 What is a combined footing? What is its primary advantage? How is it different from a grade beam?

10.16 As a very general rule, how much more settlement (maximum) can be expected under a strip footing than a square footing having the same width and contact pressure in the same soil mass?

10.17 What is the effect on the footing when its pressure bulb is interrupted by an excavation or by a basement wall?

10.18 What is the point of caution in intermixing footing types, such as intermixing spread footings and grade beams?

10.19 What is the potential danger to the structure in placing a small lightly loaded footing adjacent to a large heavily loaded footing?

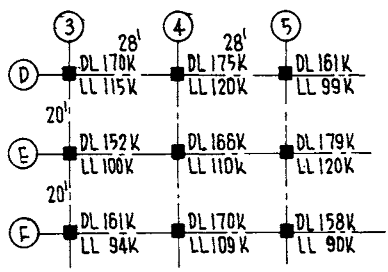
COMPREHENSIVE PROBLEMS

(Recommended for use as term problems or as part of a take-home final examination)

10-1 The layout of a group of footings somewhere in the interior of a braced frame structure is shown in the sketch.

Tributary loads at each footing are shown at each grid line. Depth of founding of all footings is 4 feet. All columns are fixed to their footings.

The supporting soil is a fine sand with a unit weight of 119 pcf and a normalized angle of internal friction of 32°. The site is subject to spring flooding. The log of Dutch cone resistances is given beside the layout sketch.



DEPTH IN FEET	q_c K/FT²
4	60
8	80
12	100
16	120
20	130
24	150
28	170

For the given structure and soil conditions,

1) Select a size for a reference footing.
2) Determine p_a and p_a' for the selected reference footing, using a factor of safety of 2.5 to bearing failure. Determine p_a'' for a settlement of 1 inch.
3) In one tabulation, select the footing sizes to meet limitations in the strength of the soil.
4) In a second tabulation, select the footing sizes that will produce equal settlement of roughly 1 inch at all footings in the grid.

10-2 For the structure of Problem 10-1, the tributary loads to the ends of a shear panel lying parallel to frame line 3 at a corner of the structure are:

End column, DL = 99 kips LL = 61 kips
First interior column, DL = 130 kips LL = 77 kips
Total base shear to two such panels: V_b = 72 kips

For the given conditions:

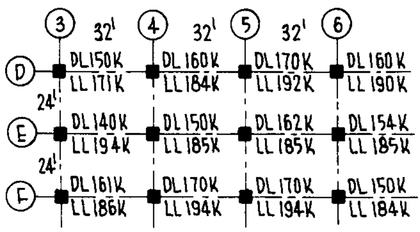
1) Determine the sizes of the two footings to meet strength limitations.
2) Determine the required sizes of the two footings to limit settlement to 1 inch

10-3 The layout of a group of interior footings somewhere in a rigid frame are shown in the sketch.

Tributary loads to each footing are listed at each grid point. Shear force at each interior footing is 2.4 kips. Center of lateral loads h_{LAT} = 24.2 feet above top of footings. Depth of founding of all footings is 3 feet. All columns are hinged to their footings.

The supporting soil is a coarse sand weighing 124 pcf with a normalized angle of internal friction of 31°. The site is not subject to water intrusion.

The log of Dutch cone resistances is given in the tabulation accompanying the sketch.



DEPTH IN FEET	q_c K/FT²
4	90
8	110
12	120
16	140
20	160
24	170
28	180

For the given structure and soil conditions,

1) Select a size for a reference footing.
2) Determine p_a and p_a' for the selected reference footing, using a factor of safety of 2.5 to bearing failure. Determine p_a'' for a settlement of 1 inch.
3) In one tabulation, select the footing sizes to meet limitations in the strength of the soil.
4) In a second tabulation, select the footing sizes that will produce equal settlements of roughly 1 inch at all footings.

10-4 The loads tributary to the end footings in Line E of the building of Problem 10-3 are:

DL = 84 kips LL = 106 kips

Total shear along frame line E: V_b = 16.8 kips

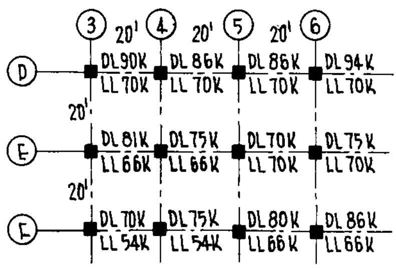
Distance between centers of end footings on frame line E is 224 ft.
Use the reference footing of Problem 10-3 to find:

1) The required size of the end footing to meet strength limitations.
2) The required size of the end footing if settlements are to be limited to 1 inch.

10-5 The layout of a group of footings somewhere in the interior of a braced frame structure is shown in the sketch.

Tributary loads at each footing are shown at each grid line. Depth of founding of all footings is 4 feet. All columns are fixed to their footings.

The supporting soil is a clay, CL, with a unit weight of 109 pcf and an unconfined compression strength of 3100 psf.



For the given structure and soil conditions,

1) Select a size for a reference footing.
2) Determine p_a, p_a' for the selected reference footing, using a factor of safety of 2.5 to bearing failure. Determine p_a'' for a settlement of 1 inch.
3) In one tabulation, select the footing sizes to meet limitations in the strength of the soil.
4) In a second tabulation, select the footing sizes that will produce a settlement of 1 inch at all footings.
5) From the foregoing calculations, select the final sizes to be used for all footings.

10-6 For the structure of Problem 10-5, the tributary loads to the ends of a shear panel lying parallel to frame line D at one corner of the structure are:

End column, DL = 54 kips LL = 39 kips
First interior column, DL = 62 kips LL = 44 kips
Total base shear to two such panels: V_b = 41 kips

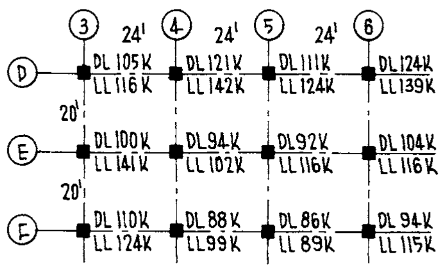
For the given conditions:

1) Determine the sizes of the two footings to meet strength limitations.
2) Determine the required sizes of the two footings to limit settlement to 1 inch

10-7 The layout of a group of interior footings somewhere in a rigid frame are shown in the sketch.

Tributary loads to each footing are listed at each grid point. Shear force at each interior footing is 3.0 kips. Center of lateral loads h_{LAT} = 19.6 feet above top of footings. Depth of founding of all footings is 3 feet. All columns are hinged to their footings.

The supporting soil is a stiff clay weighing 106 pcf, having a cohesion c of 1720 psf.



For the given structure and soil conditions,

1) Select a size for a reference footing.
2) Determine p_a, p_a' for the selected reference footing, using a factor of safety of 2.5 to bearing failure. Determine p_a'' for a settlement of 1 inch.
3) In one tabulation, select the footing sizes to meet limitations in the strength of the soil.
4) In a second tabulation, select the footing sizes that will produce a settlement of 1 inch at all footings.

10-8 The loads tributary to the end footings in Line E of the rigid frame of Problem 10-7 are:

DL = 68 kips LL = 94 kips

Total shear along Frame line E: V_b = 24.4 kips

Distance between centers of end footings on frame line E is 168 ft.
Use the reference footing of Problem 10-7 to determine:

1) The required size of the end footing to meet strength limitations.
2) The required size of the end footing if settlements are limited to 1 inch.

10-9 For the structure of Problem 10-1, it is proposed to incorporate a reinforced masonry party wall along Line 4 to separate two rental areas. The party wall will not carry any of the shear loads of the building.

It is proposed to use a grade beam to support the columns along Line 4 as well as the new party wall. The dead load of the wall is estimated to be 3.6 kips/ft; there will be no live load on the wall.

1) Determine the required size of the grade beam to support the wall and the columns, based on strength limitations.
2) Determine the settlement of the grade beam in comparison to the settlement of the reference footing.

PART IV

RELATED TOPICS IN FOUNDATION SYSTEMS

Chapter 11

OTHER TOPICS IN FOUNDATION DESIGN*

Special Design Conditions

In this chapter a few of the special circumstances that can affect the design of a foundation are discussed. Such circumstances are often completely unrelated, falling in no recognizable category. Or they may occur as an inescapable feature of the particular project, arising due to the type of soil, the location of the site, the severity of the weather, or the project design criteria.

One special circumstance, for example, is the design of buildings for which the service life is less than 10 years. For such buildings, the life is long enough to warrant a fixed foundation but not long enough to justify a permanent foundation. Some possibilities for reliable but expedient foundations for such buildings are suggested in this chapter.

Another such special topic is the design of buildings for remote locations. In such locations, the importation or production of aggregates for making concrete is often impractical. In these cases a foundation system of some nonstandard type may become feasible; a few such nonstandard foundations are discussed in this chapter.

Other topics in this chapter include foundations having irregular shapes, foundations for underpinning of buildings, foundations having high lateral loadings, exceptionally rigid foundations for stucco or plaster walls, unreinforced foundations, foundations of rubble or masonry, treated timber foundations, and foundations in expansive clays. Superficially, these topics would seem to be of interest only rarely. They appear, however, with surprising frequency.

Combined Footings

Circular, hexagonal, or octagonal spread footings are often used as foundations for tanks, silos, towers, elevated water tanks, and other free-standing structures. Although the design of foundations for tall, free-standing structures is beyond the scope of this book, the use of such foundation shapes for low free-standing

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* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

structures is well within the procedures presented earlier, provided that the resultant of loads on the footing falls well within the kern of the footprint.

Foundations for fuel tanks and water tanks up to 10 ft in diameter and located at ground level are designed in the same way as other shallow foundations; circular, hexagonal, or octagonal shapes are frequently used in such applications.
For such footings, the least width B for the design formulas is taken as the diameter of circular footings or across the flats for polygonal footings.

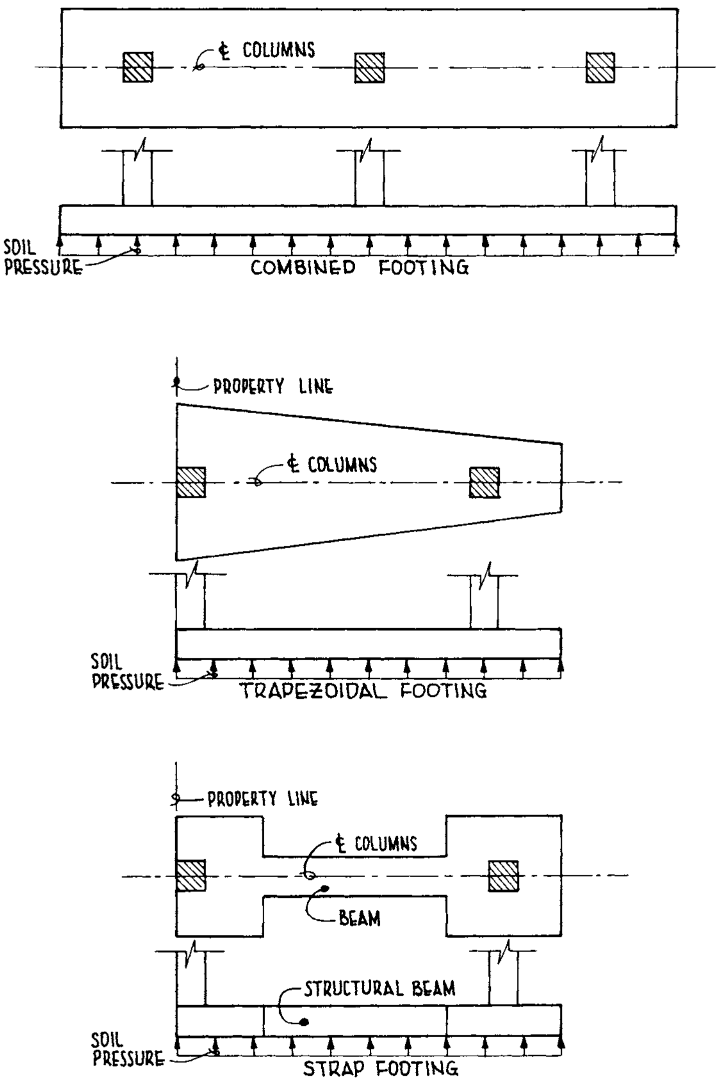


Figure 11-1 Common footing shapes.

However, the magnitude of settlement of free-standing structures is rarely limited to 1 in. The primary limitation on settlement is usually controlled by the connecting piping, which might tolerate up to $2^1/_2$ or 3 in. of settlement.

Another foundation shape that is commonly seen, especially as a combined footing, is the trapezoid. As indicated in Fig. 11-1, a trapezoidal combined footing is shown along with other configurations of combined footings. In its most common application, the dimensions of the trapezoid are set such that its centroid falls under the resultant of the two column loads, producing a uniform contact pressure at the founding line.

The following example will illustrate the design of a trapezoidal footing. The location of the centroid of a trapezoid and the area of the trapezoid, *A*, will be needed in the example; for reference, these quantities are given in Fig. 11-2.

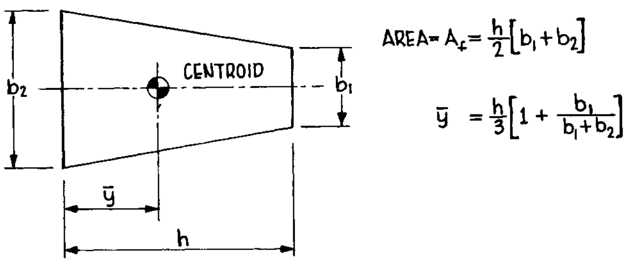
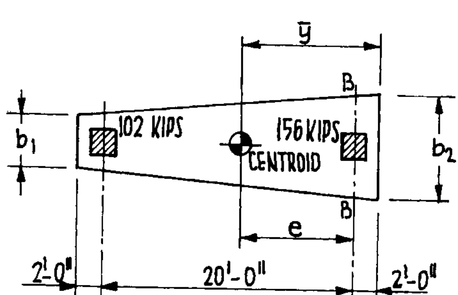


Figure 11-2 Area and centroid of a trapezoid.

Example 11-1 Design of a trapezoidal footing

Given : Column loads as shown, spaced at 20 ft o.c.
Contact pressure limited to 3000 psf

To Find: Dimensions b1 and b2 to produce uniform contact pressure.



Solution:

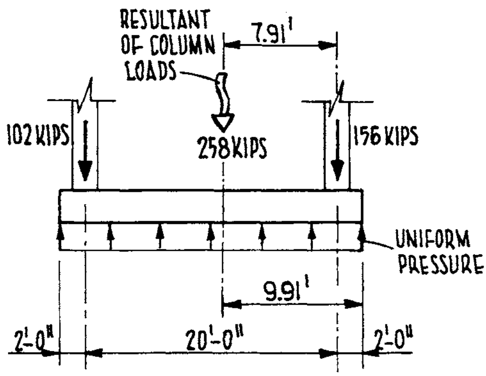
The overall length of the footing is chosen arbitrarily such that the trapezoid extends a reasonable distance beyond the two columns. In this case, 2 ft was chosen. There are thus two unknowns to be found, the widths b_1 and b_2, such that the resultant of loads falls over the centroid of the trapezoid. The magnitude of the resultant of loads is simply the sum of the two column loads,

$$R = 102 + 156 = 258 \text{ kips}$$

The location of the resultant of loads is found by simple statics by summing moments about line BB:

$$e = \frac{\Sigma Px}{\Sigma P} = \frac{102 \times 20}{102 + 156} = 7.91 \text{ ft.}$$

The resultant of loads is shown in the following sketch.



At a uniform contact pressure of 3000 psf, the required area is found routinely,

$$A_f = \frac{P}{p_a} = \frac{258000}{3000} = 86 \text{ ft}^2$$

The values of A_f and e computed above, along with the overall length of 24 ft, are substituted into the equations given in Fig. 11-2,

$$A_f = \frac{h}{2}(b_1 + b_2) \qquad \bar{y} = \frac{h}{3}\left(1 + \frac{b_1}{b_1 + b_2}\right)$$

After the substitution,

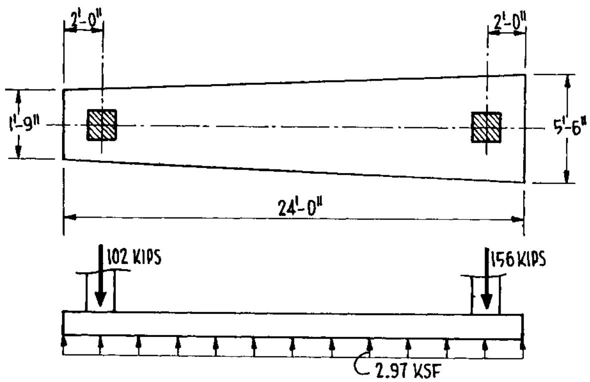
$$86 = \frac{24}{2}(b_1 + b_2) \qquad 9.91 = \frac{24}{3}\left(1 + \frac{b_1}{b_1 + b_2}\right)$$

These two equations are solved simultaneously to find

$$b_1 = 1.71 \qquad b_2 = 5.46 \text{ ft}$$

Use b_1 = 1 ft 9 in., b_2 = 5 ft 6 in.

The final results are shown in the following sketch, along with the final dimensions and pressures.



A moment's reflection will confirm that the least value that $\bar{y}$ can have in the foregoing example is $h/3$, or 8 feet, at which point the trapezoid becomes a triangle having $b_1 = 0$. Consequently, there is a very narrow range of column loads for which a trapezoid of reasonable dimensions can be selected. For any other values, the contact pressure will not be uniform.

Even when the contact pressure is uniform, however, the trapezoid does not settle uniformly, since at a uniform contact pressure, the larger end will settle more than the smaller end. Upon reflection, one must conclude that there are few circumstances where the trapezoidal combined footing offers any real advantage over a rectangular combined footing. The trapezoidal footing has been popular for may years, however, and likely will remain popular for many more.

Another popular configuration for spread footings is the strap foundation, shown earlier in Fig. 11-1 and again with two of its variations in Fig. 11-3. The strap itself may lie in the plane of the footings or it may be a "strongback", located above or below the footings. Strap footings are also called cantilever footings or pumphandle footings.

It should be apparent that the load to be carried by each pad of the strap footing can be found by simple statics. From that point onward, the design of each pad is performed exactly like any other spread footing. The two pads can even be proportioned for equal settlements using the methods presented earlier, in the event their settlements are critical.

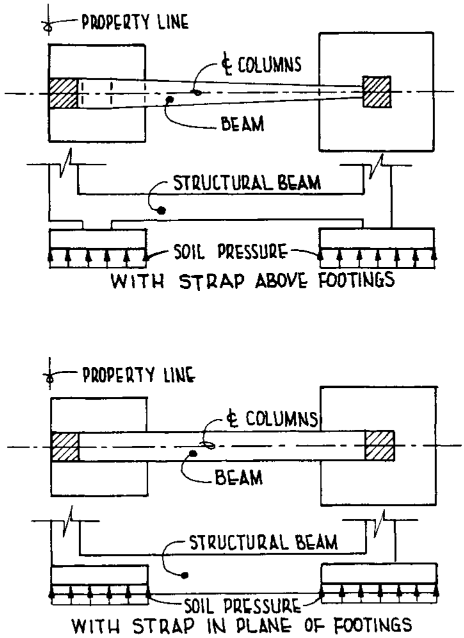


Figure 11-3 Strap footings.

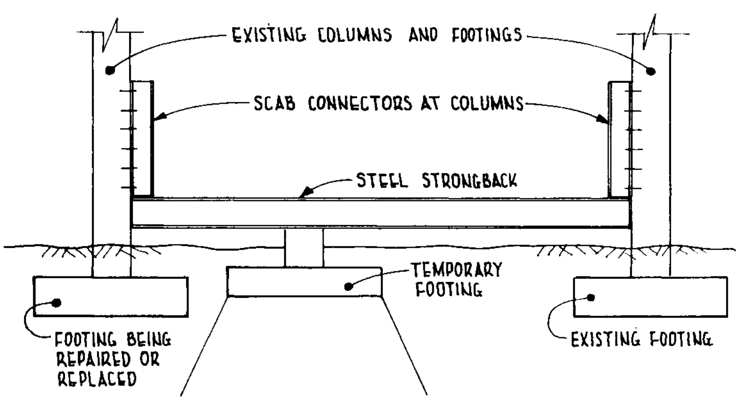


Figure 11-4 Cantilever footing as underpinning.

The cantilever footing arrangement shown in Fig. 11-3 may sometimes be useful in underpinning a building to make foundation repairs or to remove and replace an existing footing when a building is to be expanded. A simplified arrangement of such a case is shown in Fig. 11-4.

Underpinning an existing structure is one of the more difficult problems in all of foundation engineering. It should be approached cautiously, and even then only with the help of expert and experienced counsel.

By far the most common use for strap footings is in placing a column in close proximity to the property boundary, as shown earlier in Fig. 11-3. Such a case frequently occurs in downtown areas or heavily built-up areas where buildings adjoin each other. The effects of close proximity of neighboring foundations at the particular site and consequent overlapping of pressure bulbs (on the neighbor's building) must, of course, be of prime concern.

Lateral Friction Loads on Footings

The subject of lateral friction loads on footings was glossed over very lightly in earlier discussions. The building codes[21,35] generally specify allowable values for friction between the bottom of the footing and soil it contacts; these values of friction were used in earlier discussions to find the allowable lateral force on a footing. Regardless what the shape, function, or configuration of the footing was, the allowable lateral force on the footing in earlier discussions was taken at about 30% of the vertical force on the footing.

Such a simplified approach warrants further discussion. Among the more notable omissions in the approach was the omission of any consideration of the shear strength of the soil along the line of contact. Also omitted was the potential benefits due to passive pressure of the confining soil against the sides of the footing. Also omitted was the potential benefits of deliberately roughening the bottom of the footing by the addition of a "key" as shown in Fig. 11-5.

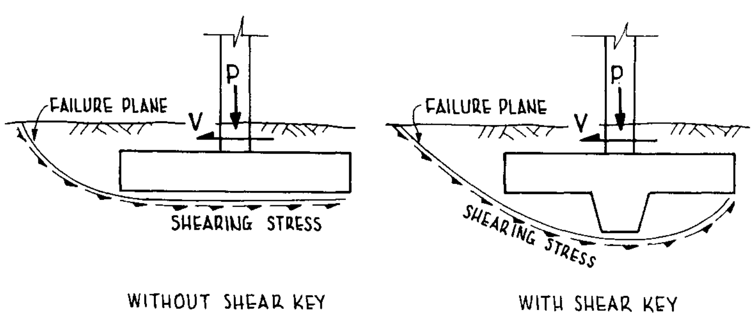


Figure 11-5 Footings with lateral load.

Two typical footings are shown in Fig. 11-5, one with and one without a shear key at its base. In both cases, the shearing force acting on the footing must be taken by some combination of friction, soil shear, or passive pressure. Failure planes are shown under both footings for an assumed shear failure in the soil and subsequent sliding of the footings.

It is apparent that the difference in total length between the two shear planes for the two cases is quite small. As a consequence, the effectiveness of shear keys in adding to the resistance of the footing is subject to serious question. The use of such keys is a holdover from older practice; it is slowly dying out but is still commonly seen.

A typical footing under lateral load is shown in Fig. 11-6, with the resisting forces shown in their approximate locations. The resisting force under the footing may be due to friction or to shear in the soil. The resisting force on the side of the footing is due to passive soil pressure.

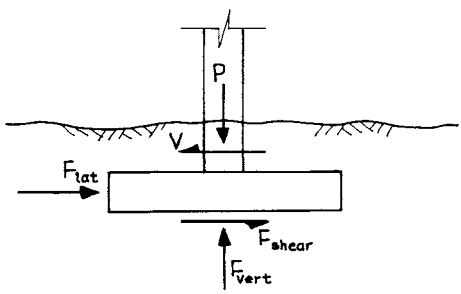


Figure 11-6 Forces on a footing.

Since a concrete footing is almost always cast directly against the supporting soil, the line between the footing and the soil is never a smooth surface but a distinctly roughened surface, even in clayey soils. Further, some penetration into the soil by the water and mortar from the wet concrete is inevitable, producing a blurred contact surface. The coefficient of friction, therefore, can be expected to be quite high.

As friction continues to increase, the limiting value of the resisting force at the contact surface becomes the shear strength of the soil, s. This limiting value of resistance can be readily derived from the Coulomb equation,

$$s = c + p\tan\phi \tag{11-1}$$

where c = cohesive strength of the soil
ϕ = angle of internal friction
p = vertical pressure

For clays, $\phi = 0$ and the shear strength of the clay is equal to the cohesion c. The cohesion c in turn was shown in Chapter 5 to be half the unconfined compressive strength at failure, q_u; hence

$$s = \tfrac{1}{2}q_u \tag{11-2}$$

The unconfined compressive strength q_u was shown in Chapter 5 to be roughly three times the allowable bearing pressure p_a; hence with a factor of safety of 3,

$$s = \tfrac{1}{2} p_a = 0.5 p_a \tag{11-3}$$

The limiting value of shear strength for clays is thus seen to be far higher than the friction value f, where

$$f = 0.30 p_a \tag{11-4}$$

Hence, sliding of the footing laterally will always govern in clay soils.

For sands, $c = 0$ and the shear strength of the sand is equal to the vertical pressure p times $\tan\phi$. The total vertical pressure (including overburden) will always be slightly more than the allowable soil pressure p_a and the angle of internal friction will almost always be more than 26°, hence $\tan\phi > 1/2$, and

$$s > \tfrac{1}{2} p_a \quad \text{or} \quad s > 0.5 p_a \tag{11-5}$$

Again, the limiting value of shear strength is seen to be far higher than the friction value, f, where

$$f = 0.30 p_a \tag{11-6}$$

Hence, sliding of the footing laterally will always govern in sands.

It is therefore concluded that for both sand and clay soils, the friction value for sliding of the footing, even though high, will always be less than the shear strength of the soil.

Refer again to Fig. 11-6. There are two resisting forces shown in the sketch, one due to the sliding friction force and one due to the passive resistance of the soil against the side of the footing. The total resistance was recognized earlier to be the sum of these two forces.

The passive resistance is rarely included as a reliable force in shallow footings; it is usually ignored. It is a viable force only where it can be assured that the overburden will be in place and undisturbed and that the concrete will be cast against undisturbed soil at the sides of the footing. Such ideal conditions are doubtful in foundation construction.

When these conditions are met, however, the design codes permit the forces to be summed to obtain the total resistance to lateral load. That, too, however, is subject to further doubt due to the inconsistency of deformations. For these forces to add, both forces would have to reach their computed maxima at the same time, a doubtful circumstance since one is a function of shear deformations and the other a function of compressive deformations.

In brief, the resistance of a footing to lateral loads can be taken safely at about 30% of the vertical load. Any refinements said to further enhance the capacity of a footing to resist lateral loads should be viewed rather skeptically.

Foundations for Stucco or Decorative Masonry

Stucco, or exterior plaster, is a popular construction material throughout much of the world. Unfortunately, it is brittle and sometimes subject to cracking. Due largely to improper construction practices, it has become associated with cheap or shoddy construction and has fallen from favor in much of the developed world.

Stucco will crack for two reasons, either through improper composition and application or through foundation settlements. The composition and application of stucco is far outside the scope of a book on foundation design; such information may be found in standard references on construction materials[20]. The practices used for design of foundations suited to stucco walls, however, are well within the scope of this book as outlined in the following paragraphs.

The design procedures presented in earlier chapters assumed that differential settlement between any two structural supports would be limited to about $^3/_4$ in. As an angular displacement, this amount of settlement on a module of 20 ft comes out to be about 0.003 rad, or $^3/_4$ in. in 20 ft. For stucco and decorative masonry, the tolerable settlement should be taken to be only about half this amount, or even less. A differential settlement not more than $^3/_8$ inch in 20 feet (10mm in 6 m) is recommended.

With such a severe limit on settlements, the use of stucco construction on clay soils would seem to be quite restricted. The surface of a clay stratum, for example, could shrink and swell more than $^3/_8$ inch in 20 feet just through the annual fluctuations in moisture content. Even clusters of leafy vegetation and their transpiration could cause enough variation in moisture content to cause this much differential movement.

As may be supposed, however, the highest variations in moisture content (and resultant shrink and swell) will occur near the surface. Moisture content in clay soils becomes much more stable just a few feet below the surface; the potential for shrink/swell can be sharply reduced by placing the foundations at these lower levels. It should not be inferred that differential movements would become negligible, but it is entirely possible that they could be reduced to half that at the higher levels.

Where stucco is being considered as a possible construction material, the limits on differential settlement should be stated at the time the soils investigation is begun. The soils report can then include estimates of the founding depths where differential settlements could be reduced to one-half the usual allowance or even less. With such information, a reliable foundation for supporting stucco or decorative masonry can be designed quite effectively.

For the sake of the preliminary design, the moisture content of clays having a plasticity index of 30 or less becomes reasonably stable at about 8 ft. The variation is not drastic, however, even at 5 ft and a depth of founding of 5 ft is suggested as a reasonable starting point for initial estimates. For heavier clays, a depth of founding up to 12 ft or even 15 ft may be necessary. These general figures apply to average years; effects of a 50-year drought or a 50-year flood could produce drastic differences in these estimates.

Stucco walls or decorative masonry at grade should be supported directly on deep grade beams rather than on strip footings. The grade beams should be continuous, with continuous reinforcement both at the top and bottom. Rigidity of the grade beam is of primary importance and rigidity comes with depth of the member; a grade beam whose depth is less than $L/16$ should not be used to support stucco, where L is the length between 90° turns in the grade beam.

The foregoing discussions apply to clay soils. The same concepts apply to sandy soils, but in general such problems are not as severe in sandy soils as in clay soils. For either soil, the determination of the minimum depth of founding for such walls should be made a part of the soils investigation; presumptive values should be avoided.

Unreinforced Foundations

Over the past 60 years, the use of reinforcement in concrete structures has increased to the point where an unreinforced concrete member is today a rarity. Where a member can be configured such that no tension occurs in it, however, no purpose is served in reinforcing it. Concrete footings are one example of members which can be shaped such that no reinforcement is required.

Where goundwater (or porewater) contains both sulfur and salt, the use of sulfur-resistant cement will often provide the necessary resistance to sulfate attack in the portland cement. The sulfur-resistant cement, however, has a lower threshhold of protection for the reinforcement, which is then subject to attack by the chlorides. The real risk to the concrete member would then be due to chloride attack on its reinforcement.

For applications in regions of high sulfur or high chloride environment, the elimination of reinforcement provides a simple but effective remedy. Fortunately, spread footings and strip footings lend themselves readily to such a solution. For grade beams, however, the elimination of reinforcement is not usually practical.

Typical unreinforced spread footings are shown in Fig. 11-7. The first is shown with stepped sides, the second with sloped sides. The stepped sides are easier to form and cast but require slightly more concrete. With the 45° slope shown at the sides of the footing in Fig. 11-7, there is no tension in the concrete and no need for reinforcement except for occasional dowels.

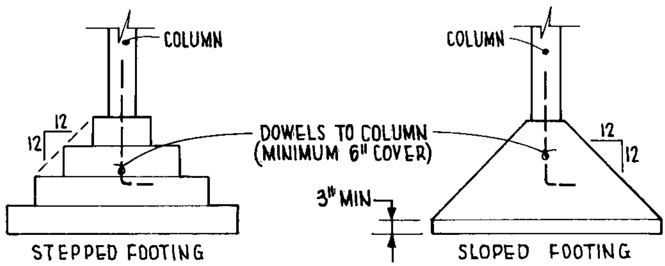


Figure 11-7 Unreinforced spread footings

Although sulfur-resistant cement has a reduced level of protection for reinforcement, the overall level of protection for dowels can be improved by increasing the cover to 6 in. as shown in Fig. 11-7. The dowels to the column can be expected to be adequately protected where sufficient cover is maintained. Except for requiring more bulk concrete, the configurations of Fig. 11-7 perform identically to the more familiar rectangular concrete pads. There is no difference at all in the way they are used. All the limitations and interactions in performance that occur in reinforced footings occur in unreinforced footings.

Unreinforced strip footings are designed in the same way as unreinforced spread footings. A typical example is shown in Fig. 11-8. Again, sides may be sloped or stepped as preferred.

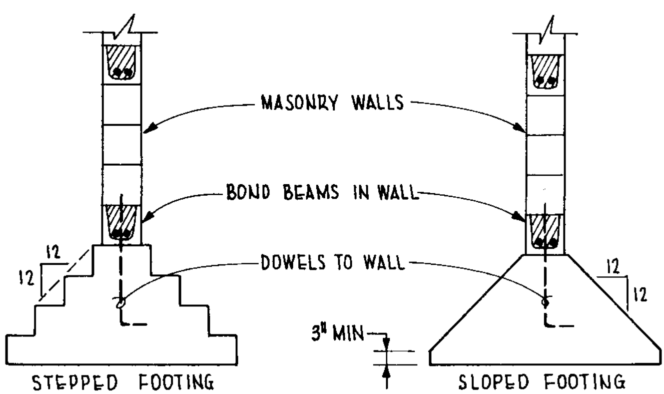


Figure 11-8 Unreinforced strip footings.

Whether reinforced or unreinfored, a strip footing is not intended to carry any flexure along its length. The wall itself must therefore be designed to sustain any variations in load that occur along the length of the wall. The bond beams shown in Fig. 11-8 are the usual means to reinforce the wall for such variations along its length.

Rubble or Masonry Foundations

There are many buildings in service today that are more than 100 years old. Almost all these older buildings that are built on shallow foundations are built on unreinforced masonry or rubble foundations; portland cement concrete was simply not in common use a hundred years ago. The use of unreinforced masonry foundations can therefore be accepted as an established and proven practice.

Masonry foundations are built in the same way that unreinforced concrete foundations are built; the only difference is in the allowable compressive strength. Some typical examples are shown in Fig. 11-9. While such foundations have faded from use in recent times due to labor costs, they remain a completely workable and practical foundation.

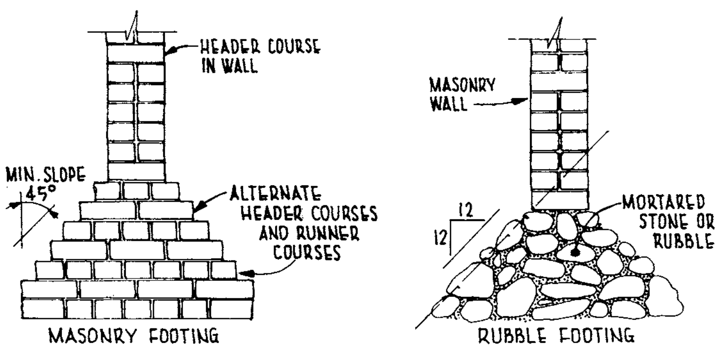


Figure 11-9 Masonry or Rubble Footings.

Rubble foundations are constructed from used or broken masonry or from fragments of stone. The rubble must of course be physically sound, even if stained and unsightly. Rubble foundations are no different structurally from other masonry foundations.

Very often, the rubble is not even mortared; the broken stones and masonry are manually placed and fitted without mortar to produce a tightly keyed masonry footing. The mortared bearing wall is then begun at the top of this unmortared keyed wall. The labor cost in building such a foundation is usually prohibitive in developed countries.

Masonry foundations built with lime mortar are still being built. Where concrete is expensive and labor is cheap, masonry foundations have traditionally been popular. They are also distinctly practical at remote locations, or where concrete is simply not available.

Treated Timber Foundations

Creosoted timber piles and creosoted railroad ties have a long and remarkable history as durable and successful foundations. Except for these two, there are few other historical examples that could be cited where timber foundations have been used. In earlier years, timber foundations simply could not endure the termites and decay organisms that accompany direct contact with earth.

In recent years, however, the development of effective chemical preservatives has made treated timber a potentially useful material for the construction of shallow foundations. To date, the use of treated timber foundations has largely been restricted to special circumstances where loads are light and alternatives are few. Nonetheless, the performance of such foundations has proven to be satisfactory and treated timber foundations can now be regarded as simply one more contemporary foundation system.

In its most common configuration as a shallow foundation, the treated timber foundation is a load-bearing stud wall sheathed with treated plywood on one side only; a typical section of such a wall is shown in Fig. 11-10. The wall is founded at some distance below grade on a treated timber plate. It is extended far enough above grade (about 12 to 24 in.) to permit untreated timber to be used throughout the building above it. Fasteners may be wrought iron, brass, or plastic, depending on the corrosiveness of the soil.

An early application of such a treated timber foundation is in the Great Lakes area, where the "all-weather wood foundation" is promoted by the manufacturers of the preservatives[30]. Its biggest advantage, as its name implies, is that construction can be undertaken at any time of year, even in winter when concrete foundation work is difficult or impossible. The acceptance of the all-weather wood foundation by mortgage companies (and federal load agencies) for ordinary 20-year or 30-year mortgages attests to its durability and reliability.

Municipal building codes[21,35] do not permit masonry to be supported by timber. Consequently, the treated timber foundation is at present suitable only for wood frame construction. Where the foundation wall can be made rigid enough, however, it could also be suitable for stucco or exterior plaster.

The rigidity of the foundation wall is of course dependent on its depth. The depth can be as much as 8 ft, since the treated plywood sheathing comes in length of 8 ft; with proper blocking the depth could be made even deeper. With such depths, the foundation wall can be made extremely rigid at a relatively nominal cost, even to the point of being rigid enough to support stucco or exterior plaster.

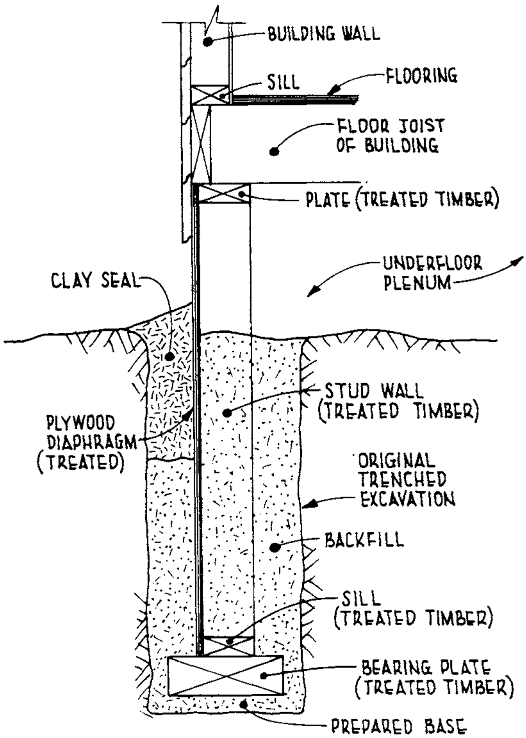


Figure 11-10 Treated timber foundation.

The structural design of the load-bearing stud wall is no different from the design of any other load-bearing stud wall. The design might even include basement walls, where the studs are designed to take lateral earth pressure as well as vertical loads. Such routine exercises in timber design are not included here.

Excavation of the foundation trench may be accomplished by simple trenching machines where the soil is suitable for such trenching. Once leveled and backfilled, the foundation wall is unaffected by weather. The modern preservatives used for treating timber are not leached out due to immersion in groundwater.

Under examination, the treated timber foundation is seen to be little more than a variation of the ordinary strip footing. As such, it may be considered for any light frame construction where a strip footing is a feasible foundation system. Its possibilities for use in remote locations are obvious, as well as in other locations where concrete is simply not available.

The treated timber foundation might also be considered as an easily constructed expedient foundation, suitable as a foundation for temporary construction. The fact that the foundation is classed as permanent rather than temporary might be a factor in its favor at times. It can also be designed for easy removal should conditions require it to be removed at a later date.

It has already been noted that a solution of salt and sulfur can be a serious hazard to reinforced concrete foundations. It should also be pointed out that this same solution is a natural preservative for timber.

The treated timber foundation is thus a contemporary innovation in foundations. It has no history of failures, however, to warn the designer of those practices which are to be avoided. Considerable reflection is therefore advisable before using treated timber foundations; it should also be remembered that the integrity of the foundation is dependent on the integrity of conventional timber fasteners.

Foundations on Expansive Clays

In Chapter 1 it was stated that the design procedures developed in this text would be applicable to "ordinary" soils, whatever that means. Expansive clays, also called active clays, were excluded from those ordinary soils; the procedures in this book should not be applied to expansive clays. Expansive clays are so common, however, that a brief discussion of their properties is warranted, to include some general discussions on the design of foundations in such soils.

Expansive clays have the distinct characteristic of shrinking and swelling as their water content decreases and increases. Magnitudes of vertical movement at the ground surface can be quite large, as much as 2 or 3 ft annually in more severe cases with several inches being relatively common in less extreme cases. The shrink-swell phenomenon is a constant process, responding almost constantly to rainfall, leaky sewer lines, water-line breaks, dry periods, heat waves, and other common and frequent events that can affect the moisture content in the subgrade around and under a building.

Movements of such magnitudes can be highly destructive to buildings, in which deflections are typically considered to be large if they are even as much as 1 inch. The movements in an expansive clay are far more than routine buildings could ever sustain; the existence of such soils on a building site obviously represents a serious hazard.

The occurrence of expansive clays is not rare; in fact, such clays are quite common. In Mississippi, for example, almost 75% of the state is covered by such clays, with activities ranging from mild to extremely severe[29]. The overconsolidated permian clays of Oklahoma and the montmorillonite soils of Wyoming are also well known as problem clays. Many other states show similar streaks of expansive clays, with varying degrees of activity and associated problems.

As in so many other areas of soils, there is little agreement on what constitutes an expansive clay. In very general terms, the activity of such clays is indicated by the plasticity index, PI. A soil having a PI greater than about 25 is usually considered to be potentially active; this rather arbitrary but simple indicator is accepted here. As the PI increases above 25, the activity and shrink-swell potential increases markedly.

In an earlier section, the concept was introduced that placing a foundation at a lower elevation where the moisture content is more stable will help to reduce deflections and settlements. That concept is also valid for foundations in expansive clays. The drawback in using this approach in expansive clays is that the moisture content may undergo rather sharp and rapid changes at depths up to 12 to 15 ft or even more. The cost of placing footings below such depths can often be prohibitive; some means to use less expensive shallow foundations adapted to these expansive clays therefore becomes a highly desirable alternative.

The reason for the rapid variations in moisture content at such depth lies in the fissures and cracks that exist in weathered clays, as shown in Fig. 11-11. These fissures occur due to the repeated wetting, drying, shrinking, and swelling of the clay as it weathers. The heavier or more plastic these clays are, the deeper, the more numerous, and more widely disseminated these fissures become.

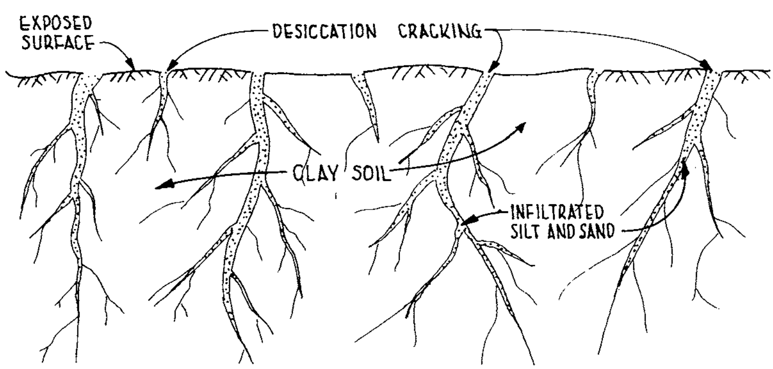


Figure 11-11 Dessication cracking

The extreme drying (and fissuring) of clay soils is termed desiccation. When the clay is dry and the fissures stand open, they gradually fill with silt and sand, thereafter providing a permanent but nonuniform network of access routes for later penetration by surface water. With repeated cycles of wetting and drying, the fissures become larger and deeper, the silt deposits become thicker and deeper, and the dissemination of surface water becomes faster, deeper, and more throughly dispersed.

For many years there was a belief that when a desiccated clay was covered by a building or a pavement, the evaporation of the porewater at the surface would be drastically reduced. Due to this reduced evaporation rate, the groundwater from below, rising by capillarity into the covered soil, would not evaporate as quickly as before. The water content in the clay would therefore increase sharply, contributing to the swelling of the clay. It was considered essential by some designers at the time to maintain an open space under the ground floor of a building where air could be circulated; the evaporation of the water could then continue, thus preventing (or at least reducing) the swelling.

More recent research has shown that there is no such continuous rise of goundwater in a desiccated clay[29,37]; the existence of the desiccation fissures thoroughly and effectively disrupts any such phenomenon. Once the water content becomes stabilized, there is no further need for concern about intrusion of water from below. The water content in a desiccated clay may therefore be maintained at a stable level simply by controlling the intrusion of surface water from above.

In any heavy clay located where shallow foundations would normally be founded, it should always be presumed that the clay will be desiccated. The problem of rapid intrusion of surface water should therefore always be presumed to be a design condition. While the rapidity of the intrusion may vary, the susceptibility of these clays to shrink-swell phenomena should never be doubted.

There are undoubtedly many ways to seal the surface of an area to prevent water intrusion. Bituminous coatings, plastic films, stabilized clay toppings, or other impermeable clay toppings spring immediately to mind. In designing any such seals or toppings, a key factor to its success would be the provision of *rapid* drainage of surface water away from the protected area (slopes of 2% or more).

In designing an impervious clay topping over the building site, it should be remembered that reworking and remixing (remolding) a desiccated clay will destroy the fissure pattern, restoring its natural impermeability. A thin surface layer of clay (10 in. or so) that has been picked up, reworked, remolded, and replaced will therefore provide a relatively impermeable barrier to surface water. With adequate drainage of this remolded layer, the undisturbed clay below this level will be relatively safe from the rapid intrusion of surface water.

Although simply remolding the clay will produce the desired result, the resulting remolded lay would again be subject to the same cycle of shrinking, cracking, and fissuring. In a short while, this thin layer will have its own pattern of fissures and its watertightness will again be diminished. If the layer were to be lime stabilized and remolded, however, its PI would be reduced markedly and its susceptibility to such fissuring would be reduced significantly.

Lime stabilization has been used successfully in such applications in highway construction[37]. It is worth considering in other applications where the clay is suitable, such as sealing a building foundation. As indicated in Fig. 11-12, the layer of remolded and stabilized clay may be placed some distance below the surface, permitting the usual landscaping and grading to be carried out.

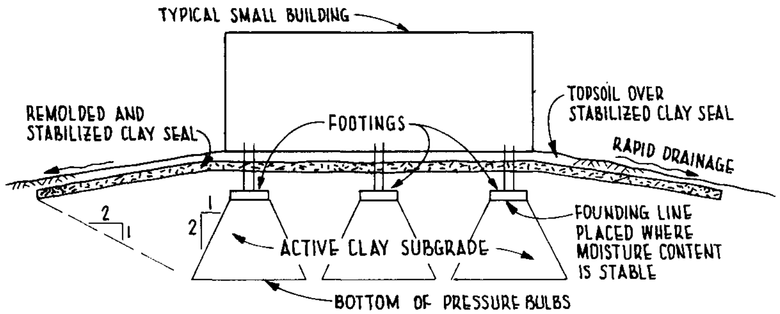


Figure 11-12 Typical surface seal.

It should not be inferred from the foregoing discussions that no water will penetrate the remolded stratum and percolate downward. Such weathering is of course inevitable. It will be slowed, however, and the clay at the founding line will maintain a more stable water content which was the initial purpose for providing the seal. The rapid drying and wetting cycle with its destructive nonuniform heaving can thus be reduced to manageable levels.

Even with the seal at the surface, however, the founding line must still be placed down at some lower level where the water content is stable. But with the remolded layer protecting the stratum below from rapid changes in the available water supply, the water content can be expected to be stable at much higher elevations than would otherwise be possible. Consequently, the cost of the foundation may be reduced significantly through the use of such features in the design.

The foundation design cannot anticipate later events that might provide an uncontrolled source of water. Typical of such events are sewer leaks, water pipe breaks, spring flooding, deliberate penetrations of the seal to plant trees or shrubs, as well as any number of other very real hazards. Such possibilities can only be weighed, judged, and countered by the individual designer as they may apply to a particular project.

Review Questions

11.1 Where might a trapezoidal footing be useful?

11.2 What is the difference between using a pumphandle foundation and using two ordinary spread footings?

11.3 What effect does a "key" have on footings that are subjected to lateral loads?

11.4 A coefficient of friction of 0.3 is comparatively high. How is such a high coefficient justified in a design procedure that is as conservative as foundation design?

11.5 Why is the passive pressure of the soil against the sides of a footing usually ignored when calculating the resistance of a footing to lateral loads?

11.6 What is the prime consideration to be met in designing a foundation for stucco or decorative masonry or any other brittle continuous material?

11.7 Why is clay such a problem soil when designing for stucco or decorative masonry?

11.8 Why would unreinforced concrete be more suited to a saltwater environment than reinforced concrete?

11.9 Why is a solution containing both salt and sulfur so much more damaging than a solution containing only one of these chemicals?

11.10 What is the difference in the response of the soil to an unreinforced footing and a reinforced footing?

11.11 When an unreinforced footing is used to support a bearing wall, how are soft spots or hard spots in the underlying soil bridged by the wall and footing?

11.12 How are rubble foundations different from unreinforced foundations?

11.13 Why are timber foundations now accepted in a practice as conservative as foundation design?

11.14 Why should plastic timber fasteners even be considered when designing a treated timber foundation?

11.15 At what PI should a clay be suspected of being expansive?

11.16 Why is desiccation such a major contributor to rapid expansion in expansive clays?

11.17 How is it possible to control expansion in a desiccated clay simply by controlling the intrusion of water from above, with no regard for intrusion of water from below?

11.18 How does remolding a clay restore its impermeability?

11.19 What does lime stabilization do to clay that reduces its susceptibility to swell?

11.20 In an expansive clay, why might a slow leak in a water pipe be more hazardous than an actual pipebreak?

OUTSIDE PROBLEMS

11-1 A column load of 240 kips occurs 12 feet away from a column load of 72 kips. Select the proportions for a combined footing to carry the load of the two columns. Maximum allowable soil pressure p_a is 2500 psf.

11-2 A column in a downtown building has a design load of 74 kips, occurring 1 foot from the property line. The nearest adjacent interior column is 20 feet away, carrying a design load of 171 kips. Allowable soil pressure is limited by settlements to 2600 psf. Select the proportions for a combined footing to support these two columns.

11-3 Six columns in a state capitol building are equally spaced in a circle 36 feet in diameter to carry the load of a heavy concrete dome over the entry area. The design load on each column is 134 kips. Select the proportions for a grade beam having a hexagon shape that will act as a combined footing to support these six column loads. Select also the locations of the columns, that is, whether they are to be located at the points of the hexagon or at the midpoints of the straight sides. Allowable pressure is limited by stringent settlement criteria to 1600 psf.

11-4 Two bearing walls are 10 feet apart, forming a long corridor. One wall carries a dead load of 12 kips/foot and a live load of 7 kips/foot. The other wall carries a dead load of 6 kips/foot and a live load of 4 kips/foot. Soil pressure is limited by settlement criteria to 1500 psf. For long-term sustained loading, select the proportions for a combined slab footing supporting the two walls. On a sketch of your results, show the locations of the centerlines of the two walls on the combined footing.

Chapter 12

FIELD TESTS AND THE SOILS REPORT*

Initiation of a Soils Investigation

Typically, the field investigation for a small-to-medium-size construction project will be subcontracted to a soils engineering contractor who specializes in such work. The field crews that actually conduct the field investigations will follow precisely a work order prepared in advance for the particular site; nothing more will be added and nothing will be deleted. The results of the field work will be transmitted to the contractor's geotechnical (soils) engineer, who will then supervise the related lab tests, evaluate the final data, and prepare the soils report.

The exact scope of work to be followed in conducting the field investigations and the questions to be addressed later in the soils report are supplied in theory by the client. In practice, however, the entire soils investigation will probably be worked out between the client's structural engineer and the contractor's geotechnical engineer during the contract negotiations. In normal circumstances, the final submission of the soils report will mark the end of the soils engineer's work. When the soils report is accepted and the fee paid, the soils engineer withdraws from the project. The soils engineer cannot be called upon again (except possibly for clarifications) without renegotiation of fees.

Using the information supplied in the soils report, the structural engineer for the project will then prepare the foundation design. It is this structural engineer who earlier prescribed (in theory at least) the content of the soils investigation. It is also this structural engineer who will eventually sign the design drawings, thereby accepting full responsibility under the law both for the adequacy of the soils investigation and of the resulting foundation design.

It is thus essential to the success of the project that the structural engineer has prescribed a course of field investigations that is pertinent, complete, and relevant to the project. In practice, the structural engineer will depend heavily on the advice of the geotechnical engineer in developing the scope and content of the soils investigation. Informal advice from the geotechnical engineer will also bear

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* All units used in this chapter are Imperial (British) units. For conversion to *Systeme Internationale* (SI) units, see the conversion factors on page 1.

strongly on the final selection of the type of foundation system, although such advice may never appear in the written records. A close working relationship between these two engineers is the best insurance possible that a workable, economical, and satisfactory foundation system will evolve.

Foundation failures are frequently costly and irreversible. (Photo courtesy of U.S. Army Corps of Engineers.)

Typically, the budget for the site investigation and soils report will not be generous; more likely, it will be marginal. As the reputation of soil mechanics has improved over recent years, budgets for site investigation have improved, but few commercial builders will allow much money to be spent poking holes in the ground. The structural engineer will have to plan the soils investigation very carefully to extract maximum information at minimum cost.

One purpose of this chapter is to outline some of the means to plan an economical soils investigation that is still pertinent, complete, and relevant to the design. A second purpose is to introduce some of the common field and lab tests that might be used in the investigation and to categorize their complexity, usefulness, and relative cost. A third purpose is to outline the required content of the soils report itself, in order that every relevant issue concerning the foundation design is addressed before the geotechnical engineer withdraws from the project.

The primary prerequisite for those who plan and conduct such an investigation is a sense of perspective. Perspective in turn is developed through experience and familiarity. Those in responsible charge of foundation work will have served a long apprenticeship in developing such perspective.

Preliminary Assessment of Site

A soils investigation intended for use in a large horizontal earthworks project such as a highway will be much different from a soils investigation intended for an isolated building with its vertical punching load. While some of the final information will be similar or even identical, the primary concern in a soils investigation for a highway is the definition of properties laterally within a strip of right-of-way; for a building, the concern is on definition of properties vertically within the Boussinesq pressure bulb. A significantly different approach is used for these two types of investigations. The methods and equipment described in subsequent sections are those commonly used to investigate building foundations; they may or may not be appropriate for other types of projects.

Undisturbed sample being taken from a manually excavated test pit. (Photo courtesy of U.S. Army Corps of Engineers)

The information to be gained from the soils investigation will include at least the following items[3,17].

Identification of the extent of stratification of the
soils within the influence of the Boussinesq pressure bulb, to include depth, thickness, strength, compressibility, and classification of each soil stratum

1. Identification and classification of any nonuniform "wedges" of soil within the influence of the pressure bulb, to include their sizes, strength, and compressibility
2. Identification of any rock formations within the limits of the pressure bulb, to include thickness, strike, dip, state of weathering, existence of bedding planes, and presence of any fault lines or outcroppings
3. Location of the groundwater table and its annual variations, to include artesian pressures, if any
4. Extent of flooding due either to natural terrain or to proximity of major drainage channels

Preliminary information concerning the foregoing items may be gained indirectly by a study of topographic maps, aerial photographs, geologic maps, and soils surveys of the area. One of the more useful sources of information is the county soils survey, prepared by the Department of Agriculture; it is usually available at the county level through the Soil Conservation Service. Another valuable source of current information as well as design data is the city/county engineer, who can usually provide up-to-date information about the area surrounding the site as well as background information about potential problem soils in the vicinity. A third very valuable source of information are the local residents, who are probably the best source of unpublished information on cracked plaster, flooded basements, cracked masonry, and other symptoms of problem soils (or poor foundation design). Local residents can also provide solid information on the extent of local flooding. Just a few telephone calls to government officials such as those named above and to local residents in the general area can often provide a wealth of preliminary information concerning a particular site or a general area.

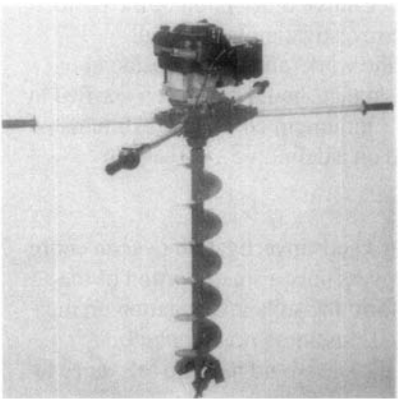
Having assimilated the available preliminary information from maps and soil surveys, the structural engineer should personally visit the site and make an assessment. Such a visit is not always feasible in today's practice, where the design office may be several hundred miles from the building site; travel budgets on smaller projects will not always permit such travel. Admittedly, the part of the site that the structural engineer wants to assess is below grade and inaccessible, but a general appraisal of the site and the topography wil be a valuable asset later when prescribing the on-site soils investigation.

Scope of the Site Investigation

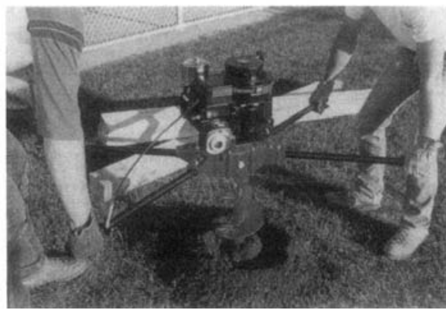
It would be impractical to attempt to do the conceptual architectural designs for a large building without consulting the structural engineer. The structural engineer, in turn, could not make any authoritative recommendations on the structural system without reliable information about the foundation conditions. As the conceptual design is progressing, therefore, one of the first responsibilities of the structural engineer is always to determine the foundation conditions, which usually will include awarding the contract for the soils investigation and report.

On smaller projects, however, the structural engineer may not even be engaged (or assigned to the project) until the conceptual architectural designs have already been completed and approved by the owners. Such a procedure is not unusual. Many, if not most, architects are perfectly capable of preparing a preliminary design, to include basic structure, on small-to-medium-size projects. With experience, the architect will have learned to allow for a considerable amount of latitude within the structural system in order that options remain open. In such circumstances, one of the first jobs of the structural engineer, when finally assigned to the project, is to determine exactly the foundation conditions as they affect the structure.

For almost all imaginable circumstances, one of the first problems to be resolved by the structural engineer is always to determine the foundation conditions. At these early stages, it would seem that the selection of the final structural system would have to await verification of suitable foundation conditions. With experience, however, and with a general knowledge of the soils and geology in a given area, the structural engineer can usually proceed with a tentative structural design with some degree of confidence that it will be suitable.



Portable Powered Auger
(Photo courtesy ELE / Soiltest)



The soils engineering contractor who is to conduct the soils investigation will have to be supplied with preliminary design information and background information. Such information will include:

1. Site plan, to include grading, earthworks, and drainage plans as available
2. Building footprints for every structure in the project
3. Overall average uniform pressure exerted on the footprint by each building in the project
4. Type of structural system used in each building
5. Foundation layouts, showing spread footings, strip footings, and grade beams
6. Location and estimated magnitude of lateral loads to be carried by each footing
7. Maximum and minimum column loads
8. Location and height of retaining walls and basement walls
9. Anticipated limits on settlements expected to be used in the structural calculations
10. Anticipated shearwall drift (angle of rotation) expected to be used in the structural calculations
11. Any isolated concentrated loads that are expected but are not included as a part of the column loads
12. Whether surcharge loading might be used to reduce settlements

Once the foregoing list of items has been supplied to the soils contractor, a program of sampling and testing that relates specifically to the proposed design can be established. The purpose, of course, is to reduce contingency; as a contingency is reduced, costs are reduced. The program of testing can then include every test that is relevant to the particular project design and exclude those that are not.

Failure to provide the soils contractor with a definitive description of the project will require that the soils contractor extend the investigation to cover all reasonably possible cases, thereby increasing the work (and cost) considerably. Even then, there may be items in the final design that should have been verified by soils tests but simply were not anticipated. For minimum cost and maximum relevance, the soils investigation must be based on a definitive and realistic preliminary design.

There is little point, for example, in performing a soils investigation over an entire site if it is known that the building footprint covers only a small portion of the site. In such a case it is necessary only to perform the soils investigation on the soils within the general area of influence of the Boussinesq pressure bulb. A subsurface investigation of soils elsewhere on the site would have no relevance to the design of this particular building.

Sand Cone test for in-place density of a fill area. (Photo courtesy U.S. Army Corps of Engineers)

Similarly, having the in-place dry density of the soil would not seem to be necessary for the sake of the structural design. However, in the event the foundation material is disturbed during construction operations, it will be necessary to know its original dry density in order to control the inevitable recompaction. Determination of dry density during the soils investigation is convenient and inexpensive; later having to determine it could produce an exasperating and expensive delay.

To establish the size of the area that must be investigated, it is necessary only to establish the extent of the pressure bulb. A typical case is shown in Fig. 12-1, where the three-dimensional pressure bulb is shown for the length and width of a rectangular building. It does not matter that the load is actually delivered to the soil by discrete footing loads rather than as an overall uniform pressure; the overall dissemination of pressure and the overall size of the bulb of influence are the same for either case (See Fig. 5-18 and related text).

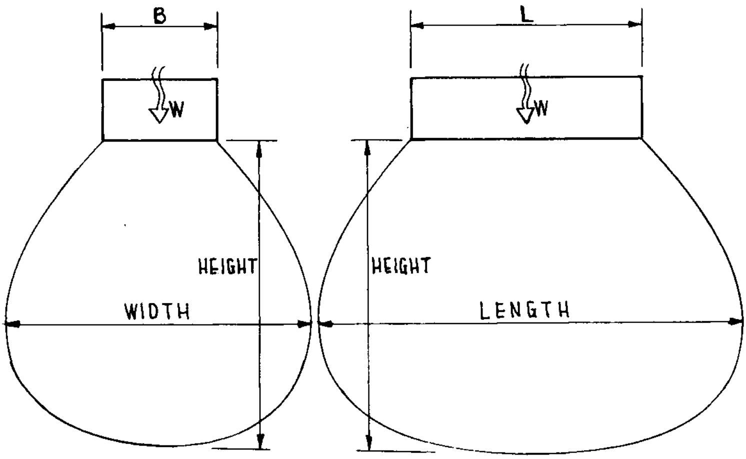


Figure 12-1 Size of pressure bulbs.

The outer pressure contour shown in Fig. 12-1 is the contour representing an increase in vertical pressure amounting to 10% of the average contact pressure p_{avg}. Within the usual limits of accuracy in soils, the influence of the building load outside this 10% pressure contour is minimal and can be ignored. (The depth and size of these pressure bulbs are developed in Chapter 5 and are shown in Fig. 5-15 for an infiinitely long strip and for a square. The solution shown in Fig. 12-1 for the size of the bulb is valid regardless of scale; B can be either a footing width or a building width, but p_{avg} is the *average* pressure over the width B.)

The Boussinesq solution presented in Chapter 5 is not extended to rectangles, which is the usual shape of a building in plan. An approximate solution for rectangles, accurate enough for planning the soils investigation, can be developed by taking the side slopes of the pressure bulb at approximately 2:1, as indicated in Fig. 12-2. The result is a truncated pyramid with a pressure increase at its base taken to be 5% above the *in-situ* pressure.

The total weight of the building in Fig. 12-2 is denoted $p_{avg}BL$. At the base of the pressure bulb, the *in-situ* vertical pressure is gH; hence, at 5% increase,

$$0.05\gamma H = \frac{p_{avg}BL}{(B+H)(L+H)} \qquad (12\text{-}1)$$

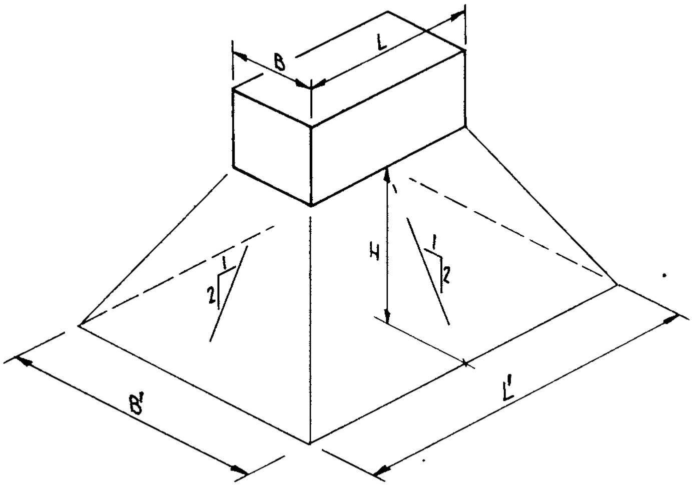


Figure 12-2 Approximate extent of influence.

Note that the 10% increase in vertical pressure used in Chapter 5 is based on the contact pressure at the top of the bulb. In Eq.(12-1), however, the 5% increase refers to an increase above the vertical *in-situ* soil pressure at the bottom of the bulb, a significantly different concept.

The ratio *L/B* is now denoted as *n*, and g is taken at 100 pcf. With these additions, Eq.(12-1) is solved for *H/B*, yielding a cubic equation:

$$\left(\frac{H}{B}\right)^3 + (n+1)\left(\frac{H}{B}\right)^2 + n\frac{H}{B} - n\frac{p_{avg}}{5B} = 0 \qquad (12\text{-}2)$$

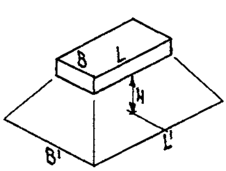
The solution of this cubic equation is given in Table 12-1 for typical values of *L/B* and p_{avg}/B. The resulting values of *L'*, *B'*, and *H* are the outermost dimensions of the pressure bulb that would occur under a rectangular building of width *B* and length *L*.

The value of p_{avg} in Table 12-1 can best be estimated from the uniform dead and live loads at each floor and at the roof. Dead load per floor on a concrete building can be estimated at about 75 to 150 psf. For steel or timber, the weight is about 60 to 80 psf. Live load is commonly between 50 to 150 psf for floors and 20 to 80 psf for roofs. As a first estimate for a concrete building, the total average pressure p_{avg} might be estimted at about 175 psf per floor plus 150 psf for the roof; for steel or timber, deduct about 50 psf from each of these.

TABLE 12-1

SIZE RATIOS OF THE APPROXIMATE PRESSURE BULB EXPRESSED AS MULTIPLES OF THE LEAST WIDTH B*

		Length-width Ratio L/B									
p_{avg}/B	Size Ratios	1	2	3	4	5	6	7	8	9	10
	L'/B	1.3	2.3	3.3	4.3	5.3	6.3	7.3	8.3	9.3	10.3
2	B'/B	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
	H/B	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
	L'/B	1.4	2.5	3.5	4.5	5.5	6.5	7.5	8.5	9.5	10.5
4	B'/B	1.4	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	H/B	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
	L'/B	1.5	2.6	3.6	4.6	5.6	6.7	7.7	8.7	9.7	10.7
6	B'/B	1.5	1.6	1.6	1.6	1.6	1.7	1.7	1.7	1.7	1.7
	H/B	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7
	L'/B	1.6	2.7	3.7	4.8	5.8	6.8	7.8	8.8	9.8	10.8
8	B'/B	1.6	1.7	1.7	1.8	1.8	1.8	1.8	1.8	1.8	1.8
	H/B	0.6	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8
	L'/B	1.7	2.8	3.8	4.9	5.9	6.9	7.9	8.9	9.9	0.9
10	B'/B	1.7	1.8	1.8	1.9	1.9	1.9	1.9	1.9	1.9	1.9
	H/B	0.7	0.8	0.8	0.9	0.9	0.9	0.9	0.9	0.9	0.9
	L'/B	1.8	2.9	3.9	5.0	6.0	7.0	8.0	9.0	10.0	11.1
12	B'/B	1.8	1.9	1.9	2.0	2.0	2.0	2.0	2.0	2.0	2.1
	H/B	0.8	0.9	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.1
	L'/B	1.8	3.0	4.0	5.1	6.1	7.1	8.1	9.1	10.2	11.2
14	B'/B	1.8	2.0	2.0	2.1	2.1	2.1	2.1	2.1	2.2	2.2
	H/B	0.8	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.2	1.2
	L'/B	1.9	3.0	4.1	5.2	6.2	7.2	8.2	9.2	10.2	11.3
16	B'/B	1.9	2.0	2.1	2.2	2.2	2.2	2.2	2.2	2.2	2.3
	H/B	0.9	1.0	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.3
	L'/B	1.9	3.1	4.2	5.2	6.3	7.3	8.3	9.3	10.3	11.3
18	B'/B	1.9	2.1	2.2	2.2	2.3	2.3	2.3	2.3	2.3	2.3
	H/B	0.9	1.1	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.3
	L'/B	2.0	3.2	4.3	5.3	6.3	7.4	8.4	9.4	10.4	11.4
20	B'/B	2.0	2.2	2.3	2.3	2.3	2.4	2.4	2.4	2.4	2.4
	H/B	1.0	1.2	1.3	1.3	1.3	1.4	1.4	1.4	1.4	1.4



* L and B are length and width of the structure
L' is outermost length of the pressure bulb
B' is outermost width of the pressure bulb
H is depth at which the increase in vertical pressure is down to 5% of *in-situ* pressure
p_{avg} is average uniform pressure in psf exerted by the structure over its footprint

Some examples will illustrate the use of Table 12-1. Remember that the results are intended only to define a general volume that will be subject to an increase in pressure. Considerable latitude may be exercised in applying the numbers.

Example 12-1 Approximation of size of pressure bulb

Given: A soils investigation is being planned for a concrete building 80 ft x 200 ft in plan, five stories high.

To Find: Estimate the size of the pressure bulb that should be used for the soils investigation.

Solution:

Since the exact floor pressues are apparently unknown, 175 psf will be used for each floor plus 150 psf for the roof. The estimated uniform pressure over the building footprint is then

$$p_{avg} = 5 \times 175 + 150 = 1025 \text{ psf}$$

For entry into Table 12-1, compute L/B and p_{avg}/B:

$$\frac{L}{B} = \frac{200}{80} = 2.5 \qquad \frac{p_{avg}}{B} = \frac{1025}{80} = 13$$

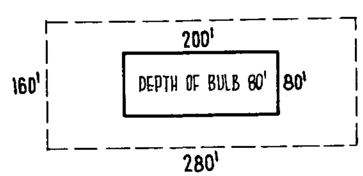
Interpolate in Table 12-1 to find

$$\frac{L'}{B} = 3.45 \qquad \frac{B'}{B} = 1.95 \qquad \frac{H}{B} = 0.95$$

For a width $B = 80$ feet, the size of the bulb that should be included in the investigation is then:

$$L' = 280 \text{ ft}, \qquad B' = 160 \text{ ft}, \qquad H = 80 \text{ ft}$$

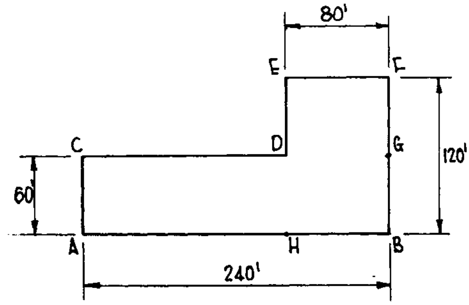
The general outline of the pressure bulb is shown in the following sketch



Example 12-2 Approximation of size of Pressure Bulb

Given: A soils investigation is to be conducted for an L-shaped building in plan, as shown in the following sketch. The building is six stories high, of steel construction.

To Find: The volume of soil that should be included in the pressure bulb in the soils investigation.



Solution:

Consider the building in two pieces, *ABGC* and *EFBH*. For a three-story steel building, a contribution of 125 psf per floor plus 100 psf from the roof will be used.

$$p_{avg} = 3 \times 125 + 100 = 475 \text{ psf}$$

For entry into Table 12-1 for the portion *ABGC*,

$$\frac{L}{B} = \frac{240}{60} = 4.0 \qquad \frac{p_{avg}}{B} = \frac{475}{60} = 8.0$$

From Table 12-1, find, for $B = 60$ ft,

$$\frac{L'}{B} = 4.8 \qquad \frac{B'}{B} = 1.8 \qquad \frac{H}{B} = 0.8$$

$$L' = 290 \text{ ft} \qquad B' = 110 \text{ ft} \qquad H = 50 \text{ ft}$$

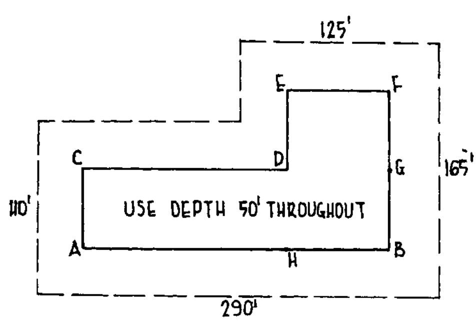
Similarly for the portion *EFBH*,

$$\frac{L}{B} = \frac{120}{80} = 1.5 \qquad \frac{p_{avg}}{B} = \frac{475}{80} = 6.0$$

From Table 12-1 find, for $B = 80$ ft, as shown in the sketch

$$\frac{L'}{B} = 2.05 \qquad \frac{B'}{B} = 1.55 \qquad \frac{H}{B} = 0.55$$

$$L' = 165 \text{ ft} \qquad B' = 125 \text{ ft} \qquad H = 50 \text{ ft}$$



Having an approximate size for the pressure bulb and a preliminary design of the structure with its foundation loads, the structural engineer will be able to outline exactly the information on soils that will be needed for the final design calculations. During the contract negotiations with the geotechnical engineer, the program of soils investigation can be specifically tailored to suit those needs. Also at these negotiations, additional items may be added to the investigation by the project architect and the project civil engineer; many such items of information that are not related to foundations might possibly be added at this time.

At this point, the general knowledge and experience of the geotechnical engineer concerning the soils in the vicinity will be an essential contribution to the proceedings. Using the same information that the structural engineer uses but with a different point of view, the geotechnical engineer may recognize problem areas in the foundation that would be beyond the narrow scope of knowledge of the structural engineer. After suitable study, he may recommend the addition of specific tests or investigations to verify that the soil is in fact adequate for the proposed design or, on occasion, to show that particular features of the design are simply not well suited to the site conditions.

It is also at this point that the geotechnical engineer can advise the structural engineer, the project civil engineer and the project architect of any potential dangers that have been found in the overall site plan. Large cuts, fills, retaining walls, and steep slopes can sometimes contribute to soil slides that could later intrude into the building foundation. Desiccated clays in the area may be subject to annual cycles of shrink and swell that could affect the integrity of the foundation. Nearby rock outcroppings can cause perched water tables and consequent water problems not only in the foundation but also in other aspects of the project design. All such site conditions outside the immediate vicinity of the pressure bulb should be investigated and evaluated by the geotechnical engineer; the resulting appraisal is then included in the final soils report.

Field samples are marked and labeled immediately in the field. (Photo courtesy U.S. Army Corps of Engineers)

The contract should always list the specific problem areas that are to be discussed and evaluated in the soils report. Items other than these specific items may be left to the discretion of the geotechnical engineer, who should be given considerable latitude in formulating and expressing such an evaluation. Wherever specific problem areas are to be addressed that will require a formal evaluation and recommendation, it is always well to name such requirements as a part of the scope of work.

Once the exact programs of soils investigation and testing (and cost) have been agreed upon, the entire scope of work can be specified and the contract can be executed. Where such a precise scope of work is specified, there will be little room for contingencies. As a result, it will be necessary to include special provisions in the contract to allow for renegotiations; renegotiations could become necessary if the field work reveals surprise conditions previously unknown or unsuspected by anyone on the project.

Such surprises are more common than one cares to admit, especially in areas where there has been little previous development. For example, the only limestone cave in the area always seems to be located directly under the proposed building rather than elsewhere on the site, or the only pocket of sensitive clay on the site will probably intrude into the pressure bulb, or the only old forgotten sanitary landfill within 30 miles will probably be located directly under this particular building site. Those who write the contract for the soils investigation will have learned long ago to expect such complications and to allow for a suitable extension of services within the terms of the contract.

Highly specialized mobile equipment is now available for soils exploration. (Photo courtesy U.S. Army Corps of Engineers)

Barring such surprises revealed by the field investigation, work on the structural design can usually be continued while the soils investigation is under way. Undoubtedly, however, there will be frequent exchanges during this period between the soils contractor's staff and the structural design staff, with the design sometimes proceeding on a "best guess" basis. Due to such exchanges, the final results of the soils investigation probably will be known to the design staff long before the actual soils report is finally received.

Field Sampling and Testing

Having agreed on the soils information to be provided and the cost to provide this information, the geotechnical engineer will set up the field and lab operations to perform the investigation. Almost certainly, the geotechnical staff will be pressed for time. In American practice, the contractor will probably have several major pieces of drilling and sampling equipment, a small staff of qualified and competent soils technicians, and a lab equipped to conduct all the more common soils tests. In less developed areas, the ultimate capability will probably be comparable, but there will be very little of the timesaving and laborsaving power equipment; the test program will probably require a great deal more time than it would in American practice.

One of the requirements of the test program will likely be to sample and classify all soil strata that fall within the Boussinesq pressure bulb. Inevitably, at least one hole will have to be drilled down through those strata in order that samples can be extracted. These holes will necessarily be comparatively deep, located within the limits of the bulb. These holes serve no purpose other than access; the cheapest possible means to drill these holes or otherwise provide access to a stratum will of course be used.

Dredged fill is particularly susceptible to dessication. (Photo courtesy U.S. Army Corps of Engineers.)

It may not always be necessary to drill to the full depth of the pressure bulb (as indicated by H/B in Table 12-1). For example, if it has repeatedly been verified that a deep stratum of a particular sand underlies the entire geologic area, there is little point in again drilling deep into the sand stratum. Once the top surface of that stratum is reached and the stratum identified, the stratum can be sampled for verification and drilling discontinued.

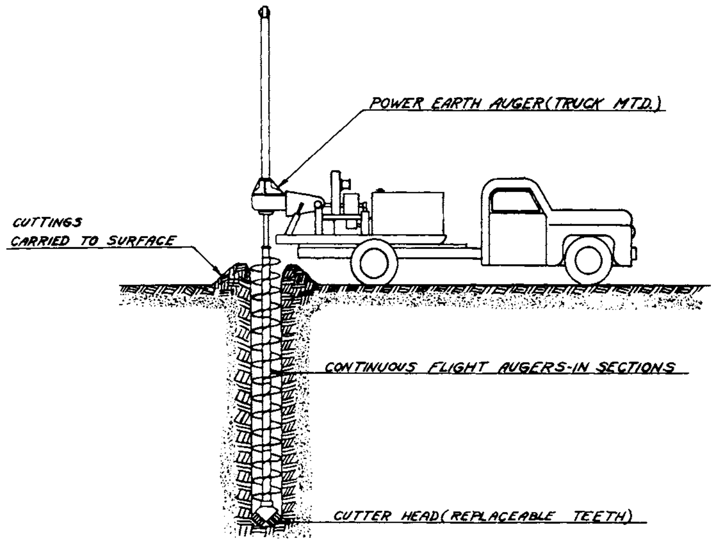
In addition to at least one deep hole drilled near the center of the pressure bulb, there may sometimes be three shallower holes drilled nearby. In plan, the three holes should form a large triangle; the shallower holes may even be located outside the limits of the pressure bulb. At each of the three holes, the depth of the first few soil strata are determined and logged. An evaluation of these depths will permit an evaluation of the levelness of the stratification, as well as identification of possible wedges of soil that do not extend across the entire building site. Sampling is not usually performed from these shallower holes; their only purpose is to permit identification of depths and thicknesses of the upper two or three soil strata.

At shallow elevations, a *probe* may be useful in locating the depth of subsurface rock or even in finding changes in soil type, if such changes are distinctive. A probe is little more than a pointed steel rod or hollow pipe, driven into the ground until it hits something that feels different. A water *jet* is sometimes used to advance the probe in hard or gravelly ground. The applications of a probe are obviously quite limited.

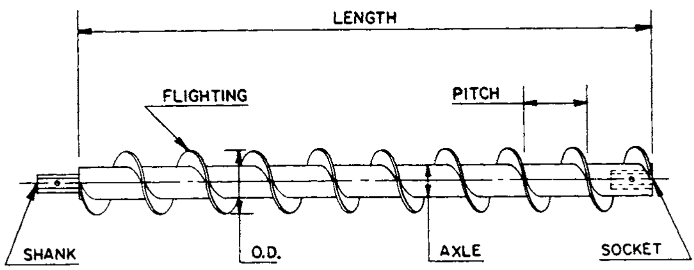
A *test pit* may be useful at shallow excavations; a test pit is a hole that is actually dug (or bulldozed) down to the founding line or even lower in order that samples can be extracted. Insofar as the sampling itself is concerned, a test pit is probably the best means of access to the stratum. Its cost is usually too high to permit depths more than about 10 ft, although some backhoes can dig up to about 15 ft.

Augers provide a low-cost means of access, useful up to a maximum depth of about 100 ft; hole size can be anywhere between 4 and 12 in. in diameter. They may be used to bring the material up for examination, but the material will be disturbed and remixed to such an extent that it is usually suitable only for classifying the sample. Once the hole is drilled and cleaned, however, undisturbed samples can be extracted from the bottom of the hole as in any other method of drilling. Manual augers can go to depths of 6 to 8 ft, small (short-flight) powered augers to about 20 ft, and large (continuous-flight) powered augers to about 100 ft.

Wash boring is a means of drilling using a water jet and agitation to advance the hole. The effluent water is a soil-water slurry and is almost useless in identifying the soil. The driller locates changes in soil type by noting changes in color and texture of the wash water effluent. At each level where a sample is required, the hole is cleaned and conventional sampling equipment is used to extract the sample.



Truck-mounted continuous flight auger

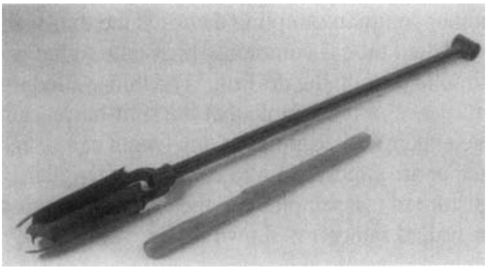


Auger nomenclature

Percussion drills, or *spudders*, literally pound a hole in the ground, using a long heavy steel bar suspended from a steel cable, having some type of drill bit attached to its base. The "tools" are repeatedly picked up and dropped about 3 ft, punching a hole in the ground. A water slurry is maintained at the bottom of the hole; the slurry water is periodically bailed out as it thickens, replaced with clean water, and spudding is then resumed. The method is cheap and can be used to drill holes up to 200 or 300 ft deep. It includes the capability of placing steel casing (a steel liner) to stabilize the sides of the hole. As with wash borings, at each level where sampling is required the hole is cleaned and conventional sampling equipment is used to extract the sample.

Rotary drills are often used when one is "pioneering" an area, that is, the geology of the locality is not well known and the soil samples must be identified throughout the entire depth of the boring. Rotary drilling utilizes a series of hollow drill rods with a drill bit attached at the bottom. Continuous sampling is possible since the undisturbed core of soil enters the hollow barrel and can then be retrieved and examined. Rotary drills have been adapted into such a variety of applications that they have become by far the most widely used type of drill in soils explorations in today's practice.

Manual operation (Iwan Type) post-hole auger used in soil sampling. (Photo courtesy ELE / Soiltest.)



As a general rule, the larger the hole diameter, the greater the cost of drilling it. Unusually small holes, however, prohibit extracting a useful-size sample. In general, a hole size 4 to 6 in. in diameter will accommodate the more common sampling equipment.

Often, a hole will have to be "cased", that is, lined with pipe, to keep it from caving in; the problem is almost certain to occur in sands lying below the water table. Lightweight thin steel casing is commonly used to case the hole. The casing can be extracted when sampling is completed and used again in later operations.

Regardless how the hole was advanced to a particular stratum, the system for sampling or testing the soil in the stratum must be carried out at the bottom of a small hole by an operator located at the surface. Over the years, techniques and equipment have evolved that permit such sampling and testing to be done with a minimum of unwanted disturbance to the soil.

A very common sampling device is the split-barrel sampler, shown in Fig. 12-3. The sampler is fitted to the end of a driving rod and lowered to the bottom of a thoroughly cleaned hole. The sampler is driven into the soil using a 140-lb hammer falling 30 in.; the sample undergoes considerable disturbance during this operation. The sampler is recovered and taken to the lab, where, upon disassembly of the sampler, the soil sample is recovered for subsequent testing.

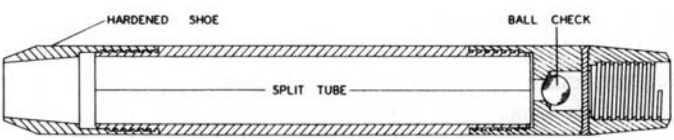
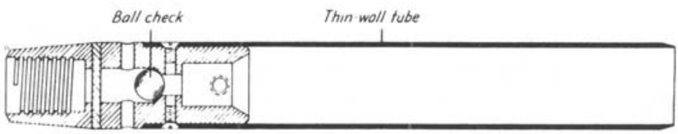


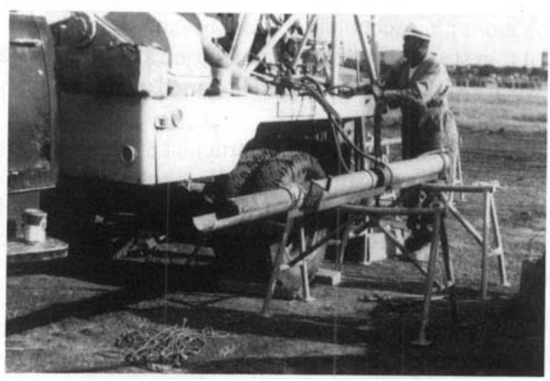
Figure 12-3 Split-barrel sampler

The number of blows required to drive a 2-in. sampler 1 ft into the soil is the standard penetration number N for that soil; the standard penetration test (SPT) is described in more detail in Chapter 5.

Another common sampling device is the thin-walled (Shelby) tube sampler. The thin-walled tube is commonly pressed into the soil rather than driven, usually by a hydraulic jack on the drill rig. The thin-walled sampler does not produce the disturbance in the sample that the split-barrel sampler does, but it is somewhat more expensive. Samples of any length can be recovered, the tube and sample together are simply cut into convenient lengths after recovery, and these yet undisturbed test samples can then be distributed in the lab as needed. A typical thin-walled sampler is shown in Fig. 12-4.



Thin-walled tube sampler



Sample being removed from tube sampler
(Photo Courtesy U.S. Army Corps of Engineers)

Figure 12-4 Thin-walled sampler

There are other samplers in common use, usually modeled after the split-barrel sampler or the thin-walled sampler, many being nothing more than a variation of some kind to solve a particular sampling problem. An example of such a variation is a retainer mechanism fitted to the split-barrel sampler. In these devices a flap or spring-steel retainer is located near the mouth of the sampler, which closes and retains the sample as it is being taken to the surface. Such a retainer is usually necessary in sands located below the water table; without it, the sample would simply fall out of the sampler as it is being withdrawn.

Another variation is that of the piston sampler, in which a thin-walled tube is fitted with an internal piston which keeps the soil under pressure as the tube is pressed into the soil. A piston sampler is shown in Figure 12-5. The recovery of undisturbed samples in a piston sampler is much improved over those in an ordinary thin-walled sampler.

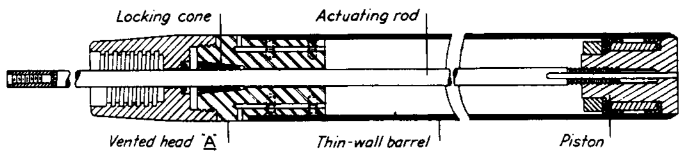


Figure 12-5 Piston sampler

There are a variety of other types of sampling devices in existence, each with its own adherents, each with its own set of advantages and disadvantages. As a rule, when complexity increases, cost increases. The user must eventually accept whatever sampling system gives acceptable results within a reasonable cost. It must be remembered, however, that the quality of sampling which might be acceptable in a foundation investigation for a small building might be completely unacceptable in a soils investigation for a dam.

For foundations, the split-barrel sampler is very commonly used in those cases where the need is simply to determine stratification, density, and classification of the soil. The blow count on the sampler yields a measure of the angle of internal friction and strength, thus permitting a reasonably complete appraisal of the soil to be made at the particular depth using only a single test procedure. Unfortunately, the SPT blow count is most accurate in sands; its accuracy diminishes rapidly as cohesion (clay content) increases. In addition, samples taken in a split-barrel sampler are classified as disturbed samples and may not be used in sensitive lab tests such as the consolidation test or the unconfined compression test.

In recent years, the Dutch cone test has grown considerably in popularity. The test is discussed in Chapter 5; it is a penetration test, directly parallel to the SPT. It may also be conducted at the bottom of a borehole, but its use does require another operation in addition to the sampling operation. For the SPT, the sampling operation and the blow count are accomplished at the same time, reducing time and cost.

As clay content increases, however, the vane shear test described in Chapter 5 becomes more accurate than the penetration tests such as SPT and Dutch cone. The vane shear test is readily conducted at the bottom of a borehole, provided that the vane is driven to a depth such that disturbances from the drilling, washing, and cleaning operations are no longer present. Having the results of both the penetration test and the vane shear test at a particular level, along with a soil classification and a knowledge of the stratification, the geotechnical engineer makes an evaluation that is accurate enough for a small-to-medium-size project.

It is again emphasized that these field tests reveal only the total shear strength of the soil. They do not provide a means to distinguish between the amount of strength that is derived from cohesion or the amount due to internal friction. Nor is there any direct assessment of settlements; the judgment of the geotechnical engineer will provide the primary evaluation of settlements when the test program is limited to these two field tests.

Where additional test information is required, test samples would be taken. These samples would be carefully identified and taken back to the laboratory for further tests. Typical of such lab tests are the direct shear test in sands, the unconfined compression test in clays, and the triaxial compression test in mixures of sands and clays. Even where the field tests are the only tests to be performed regarding the soil strength, however, laboratory testing will still be required for water content, specific gravity of solids, and in-place void ratio.

Field Load-Settlement Tests

Except for the consolidation test, none of the field or lab tests just described reveal any settlement properties of the soil. Since the consolidation test applies only to heavy clays, there is little information that can be developed regarding settlements in sands or in mixtures of sands and clays. Reliable and inexpensive methods to determine actual long-term settlements in the field have yet to be developed.

For determination of short-term settlements, however, a type of test called a load-deflection test has been in use for quite some time. A sketch showing a typical setup for such a load-deflection test is shown schematically in Fig. 12-6. There is no standard configuration or size or loading for the load-deflection test; each test is conducted however the investigator deems appropriate to suit the particular needs of the project.

In concept, the load-deflection test is quite simple. A bearing plate representing a footing is placed at a depth comparable to the founding line for the actual foundation. It is then loaded in increments, one increment being in the range of about one-tenth the estimated failure load. When an increment of load is added, deflection will be quite rapid at first, slowing markedly after only a few minutes or hours. When the deflection rate has decreased to less than about 0.005 in. in 30 min. another increment is added and the procedure repeated.

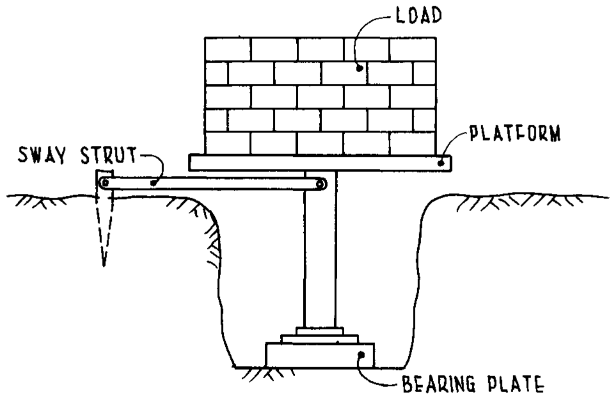


Figure 12-6 Typical setup for load-deflection test.

The results of the test are plotted very similarly to those of the consolidation test as discussed earlier. A settlement-log time curve is plotted for each increment of load as indicated in Fig. 12-7. From each of these curves, a point is chosen which best indicates the maximum short-term deflection under the particular load; this point is usually chosen as the beginning point of the final straight-line portion of the curve, designated D in the curves of Fig. 12-7a.

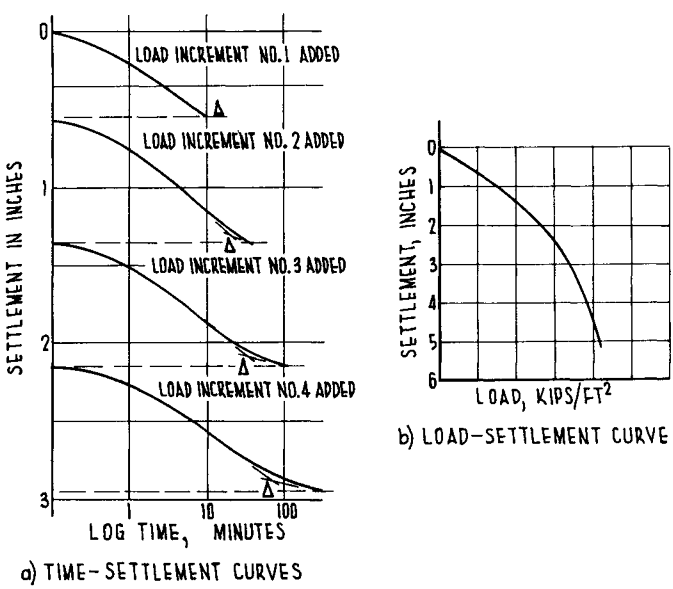


Figure 12-7 Load test curves.

The load-deflection curve of Fig. 12-7b represents the behavior of only one size of plate. The behavior of a smaller plate or a larger plate could be much different. Extrapolation of results into larger size of foundations is quite commonly done, however, especially when comparing settlements between adjacent footings.

The field load-deflection test is obviously quite slow, expensive, and cumbersome. If results are to be meaningful, the bearing plates should be large, but for large plates the required load becomes upward of 30, 40, or 50 tons. For smaller sizes of plates, say 2 ft square, loads become more manageable, but the Boussinesq pressure bulb extends such a short distance into the stratum that the results mean little. The following example illustrates this problem with "scale" in a load-deflection test.

Example 12-3 Scaling a Test Load in a Field Load Test

Given: A footing 10 ft square having an average uniform pressure p_{ftg} is assumed to have a pressure bulb extending roughly $2B$ or 20 ft below the depth of founding, producing a pressure of $0.10p_{ftg}$ at that depth.

To Find: The pressure that would have to be exerted on a test plate 2 ft square to produce the same increase in pressure at the same depth.

Solution:

The depth of infuence of 20 ft for the footing amounts to 10 x B for the 2-ft plate. From the Boussinesq pressure bulb of Fig. 5-15, the pressure at that depth is roughly $0.005p_{pl}$. This pressure is equated to that for the footing,

$$0.005p_{pl} = 0.10p_{ftg}$$

Hence

$$p_{pl} = 20p_{ftg}$$

This result indicates that the pressure on a 2 ft square test plate would have to be some 20 times the contact pressure of the real footing if it is to reproduce the Boussinesq pressure bulb and provide meaningful results.

It is doubtful whether the soil itself would be strong enough to sustain such a load, since the factor of safety on the soil is usually less than 4; the soil would most likely fail long before the necessary pressure could be attained.

One last point concerning the load-deflection test merits comment. The slope of the initial portion of the load-deflection curve of Fig. 12-6 is something like the modulus of elasticity that would be used in more elastic materials. It is, in fact, the short-term modulus of subgrade reaction for this particular soil loaded by this particular size plate located at this particular depth of founding. Whether the modulus can or cannot be extrapolated into other loading rates, into other plate sizes or into other elevations within the stratum is a matter for each foundation designer to decide.

Concerning field tests in general, one of the more notable shortcomings in field testing is the lack of a suitable test to determine field densities at in-place pressures that can be conducted at the bottom or at the sides of the borehole. The current tests for field densities require an operator; they can only be conducted therefore at atmospheric pressures in a place accessible to the operator. An inexpensive general test to measure in-place densities (or void ratios) at in-place pressures anywhere along the borehole remains an elusive point in the technology.

Common Laboratory Tests

Throughout the preceding chapters, various laboratory tests for soils have been introduced and discussed. For the sake of convenience, these tests, along with other common lab tests, are summarized in the following list. The standard procedure for conducting these tests, when such a standard procedure exists, is indicated by its test designation as given by the American Society for Testing and Materials (ASTM). In some cases there are particular requirements for field sampling that must be observed in conjunction with a particular test. Such requirements are indicated, together with the approximate size of the sample that is required for the test.

Tests for Index Properties and Soil Classification

1. *Water content:* ASTM D2216, natural moisture content protected while awaiting test, 10- to 20-g sample required for test
2. *Specific gravity of solids:* ASTM D854 and ASTM C127, 150 to 500 g of sample required, depending on coarseness of grain sizes
3. *In-place density:* ASTM D1556 or D2167, usually performed in-place as the moisture sample is being extracted, but may be made on a large (10 kg) undisturbed sample
4. *Liquid limit:* ASTM D423, performed on fine-grained soils, requires 100 to 500 g of that portion of the sample passing the No. 40 sieve
5. *Plastic limit:* ASTM D424, performed on fine-grained soils, requires about 20 to 30 g of that portion of the sample passing the No. 40 sieve
6. *Sieve analysis:* ASTM D422, performed on dried whole sample, requires up to 500 g of sample with grain sizes up to 3/8 in., up to 5000 g for larger sizes.

Tests for Friction, Cohesion, Compression Strength

7. *Direct shear:* ASTM D3080, performed on disturbed cohesionless sample about 4 in. by 4 in. square, up to 1 in. thick
8. *Unconfined compression:* ASTM D2166, performed on undisturbed cohesive soils, sample cylindrical in shape, diameter 2.8 in., length two to three times diameter
9. *Triaxial compression:* ASTM D2850, sample size and shape same as that for unconfined compression test, but sample may be cohesionless if undisturbed

10. *Vane shear:* not a standard test, usually performed in-place in the field but may be performed in the lab on a large sample having dimensions at least three times the vane size

Tests for Compressibility

11. *Consolidation:* ASTM D2435, performed on undisturbed sample of cohesive soil at least 2.5 in. in diameter, with thickness one-third to one-fourth the diameter, usually 3/4 in.

Tests for Compaction and Recompaction

12. *Modified proctor:* ASTM D1557, disturbed sample of cohesive soil up to 50 kg; permitted to reuse sample in successive compactions but preferable not to do so
13. *Maximum and minimum densities:* ASTM D2049, performed on disturbed samples of cohesionless soil, from 10 to 100 kg of sample required, depending on grain size

There are, of course, many other laboratory tests for soils than these, covering a variety of such properties as soil chemistry, soil permeability, electrical resistivity, shink-swell potential, response to cylical loading, and others. Such tests are not usually appropriate for foundations for small buildings and are not presented here. Under special conditions, however, such tests may reveal particular items of information relevant to a particular foundation design and may be justified even for small buildings; the advice of the geotechnical engineer should be heeded in such special cases.

The Soils Report

The end product of the soils investigation is the soils report. It forms the basis of all decisions to be made regarding the site throughout the remaining design stages of the project, and often is used by the construction contractor throughout the construction stages as well. The soils report must stand alone; it cannot be assumed that those who originated the investigation or who carried out the work will be available for further discussions over the next few months or years while the report is being used.

Under ideal circumstances, the soils report would be simply a statement of facts, indicating exactly the tests performed, the results obtained, and the conclusions derived. In soils, however, the conclusions must often be shaded by judgment and opinion. The soils report must point out those areas where conclusions have been subject to such judgment or bias, and must include the relevant test or background data that justify the conclusions drawn.

There will be little room for subjective narrative in the soils report. Every statement must be based on a test result or a field observation that was included in the program of investigation. Similarly, every conclusion must indicate clearly its source in order that months or years later, any new personnel assigned to the project can fully understand the basis of the design they are expected to take over and to complete.

Over the years, the soils reports prepared for smaller projects by a particular soils firm usually will have evolved into a rather rigid, condensed format. The recommendations made in the report will probably be limited to those listed in the "scope of work" in the contract. Where the conract specifies a subjective analysis on a particular subject, the narrative will usually be brief and succinct to the point of being an outline. Such is the legacy of least-cost soils investigations; the structural engineer must understand implicitly all the ramifications of the conclusions stated so briefly in the soils report.

In general, the soils report will include at least the following items, stated in narrative form:

1. Identification of the contract and its date as the basis for the work.
2. Description of the scope of work, stating notable contract limitations, if any, in certain areas of the work.
3. An executive summary of the conclusions and recommendations that were developed as a result of the investigations, to include all major findings that will affect the design.
4. Description of the project, with a reduced-size site plan, noting access routes, survey markers, or other identification and orientation points on site.
5. Summary of foundation types, column loads, lateral loads, basement walls, retaining walls, shearwalls, and other structural and architectural features as they affect the investigation.
6. Notable site features such as streams, ponds, cuts, fills, rock outcropping, and other topographic and geologic featues as they affect both the investigation and the project as a whole.
7. Description of the field soils investigation, type and manufacturer of equipment used, type, size, and manufacturer of sampling and testing devices utilized, listing of field tests run and data gathered, with a map locating all boreholes, sampling locations, and test locations. (A machinery and equipment list may be included to shorten the narrative.)
8. Listing of the lab tests run in conjunction with the soils investigation, naming the type, size, and manufacturer of test equipment where appropriate. (A lab equipment list may be included in the appendix to shorten the narrative.)
9. Findings and conclusions derived from the investigation, to include sketches showing the pressure bulbs, the extent of stratification of the soil, areas in which the investigation did not extend, index properties of each stratum as appropriate, locations of any formations that could adversely affect the design, location of water table at minimum and maximum levels, extent of reported flooding, and other relevant findings derived throughout the course

of the investigation. Pertinent test data should also be reproduced, to include *e*-log time curves, consolidation curves, load-deflection curves, filter curves, or other commonly used graphic solutions corresponding to a particular line of investigation. The interpretation of the test data and the conclusions drawn as a result of the test should always be stated specifically.

10. Recommendations based on the findings and conclusions, to include allowable soil pressures under both vertical and lateral loads and the anticipated extent of settlement under these allowable soil pressures, limitations in the strength of the soil, limitations in the estimated settlements, and means to include the effects of ground water as appropriate. Recommendations will also cover problem areas specified in the contract as well as possible remedies for other problem areas revealed by the investigation subsequent to execution of the contract. Where applicable, attention may be called to certain design features that are inappropriate to the site conditions.
11. Report of field operations, to include all boring logs, test data, and other information prescribed for the field investigation and paid for under this contract. This report and related data are usually included as an appendix to the soils report.
12. Report of lab tests, include backup lab information such as *e*-log time curves, major test items (not to include index properties), triaxial tests, unconfined compression, direct shear, and other significant test items paid for under the contract. This report and related data are also usually included in the appendix.

Review Questions

12.1 If the geotechnical engineer is not given a preliminary design by which he or she can plan a soils investigation on a building site, what is the recourse?

12.2 Name the primary items of information to be obtained in the soils investigation.

12.3 What are some of the sources of general information that can be used to make a preliminary evaluation of a construction site?

12.4 On ordinary small-to-medium-size construction projects, who is responsible to determine the foundation conditions?

12.5 Name some of the typical items to be included in the preliminary design information that could produce an impact on the nature of the soils investigation.

12.6 In conducting a soils investigation for a building, why limit the subsurface investigation to the soil enclosed by the Boussinesq pressure bulb?

12.7 Why limit the investigation to those locations where the increase in pressure will be about 5% of the existing at-rest pressures?

12.8 When estimating the overall average pressure exerted by a building over its footprint area, what is a nominal average pressure attributable to each floor? To the roof?

12.9 There will almost always be at least one deep boring in the program of soils investigation for a building foundation. If only one such deep boring is to be drilled, where should that deep boring be located?

12.10 Where only one deep boring is to be drilled in an investigation, there will often be shallower borings around the site in support of this one deep boring. Should these be in a straight line? Why?

12.11 What are the advantages and disadvantages of a probe in sounding out the changes in soil type under a building?

12.12 What are the advantages and disadvantages of using a test pit when conducting a soils investigation?

12.13 About how far can a powered auger advance a hole?

12.14 What is a wash boring? What is its primary disadvantage when used in soils investigations?

12.15 What is the most common type of drilling rig used in conducting soils investigations. Why?

12.16 Does a split-barrel sampler produce a disturbed sample or an undisturbed sample?

12.17 Does a thin-walled sampler produce a disturbed sample or an undisturbed sample?

12.18 Name the two penetration tests that are commonly conducted at the bottom of a borehole.

12.19 Name the common shear test that can be conducted at the bottom of a borehole.

12.20 In what type of soil are the penetration tests more accurate?

12.21 In what type of soil is the vane shear test more accurate?

12.22 What lab test is used to provide settlement properties in clay soils?

12.23 What is the purpose of the field load-deflection test?

12.24 What is the primary disadvantage in the use of the smaller plate sizes in the load-deflection test?

12.25 What is the primary disadvantage in the use of the larger plate sizes in the load-deflection test?

12.26 Why is so much emphasis placed on having all the information from the soils investigation written down?

12.27 Why is it necessary to have a complete list of the lab and field equipment that was used in conducting the soils investigation?

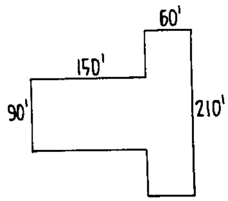
12.28 Who prescribes the information to be included in the soils report?

12.29 Who writes the soils report?

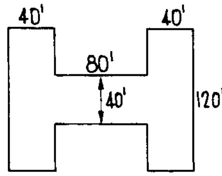
OUTSIDE PROBLEMS

Determine the size of the pressure bulb that should be used in the soils investigations for each of the given building shapes.

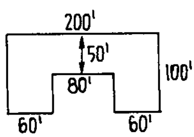
12.1 Concrete construction, four stories



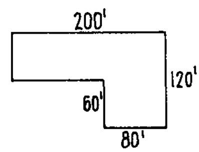
12.2 Steel construction, six stories



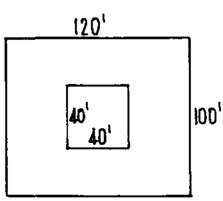
12.3 Timber construction, two stories



12.4 Concrete construction, five stories



12.5 Timber construction, three stories



12.6 Steel construction, five stories

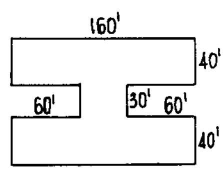


TABLE OF CONVERSION FACTORS FOR UNITS COMMONLY USED IN FOUNDATION DESIGN

To convert	To	Multiply by
inches	millimeters	25.400
inches^2	millimeters^2	645.16
inches^3	millimeters^3	16387
feet	millimeters	304.80
feet	meters	0.3048
feet^2	meters^2	0.09290
feet^3	meters^3	0.02832
feet^3	liters	28.3169
feet^3	gallons	7.48055
pounds (lbs)	Newtons (N)	4.44822
kips (k)	kilonewtons (kN)	4.44822
slugs (lb•sec^2/ft)	kilograms (kg)	14.59390
lbs/ft (plf)	Newtons/meter (N/m)	14.59390
kips/ft (klf)	kilonewtons/meter (kN/m)	14.59390
lbs/in^2 (psi)	N/mm^2 (MPa)	0.006895
kips/in^2 (ksi)	N/mm^2 (MPa)	6.89475
lbs/ft^2 (psf)	Newtons/meter^2 (Pa)	47.8803
kips/ft^2 (ksf)	kilonewtons/meter^2 (kPa)	47.8803
lbs/ft^3 (pcf)	N/m^3	157.0874
kips/ft^3 (kcf)	kN/m^3	157.0874
gallons(gal.)	liters	3.785
gallons of water	pounds	8.342
miles/hour (mph)	kilometers/hour (kph)	1.609
yards^3 (cy)	meters^3	0.76455

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