CHAPTER ELEVEN

FOUNDATIONS ON DIFFICULT SOILS

11.1 INTRODUCTION

In many areas of the United States and other parts of the world, certain soils make construction of foundations extremely difficult. For example, expansive or collapsible soils may cause high differential movements in structures by excessive heave or settlement. Similar problems can also arise when foundations are constructed over sanitary landfills. Foundation engineers must be able to identify difficult soils when they are encountered in the field. Although not all the problems caused by all soils can be solved, preventive measures can be taken to reduce the possibility of damage to structures built on them. This chapter outlines the fundamental properties of three major soil conditions — collapsible soils, expansive soils, and sanitary landfills — and methods of careful foundation construction.

COLLAPSIBLE SOIL

11.2 DEFINITION AND TYPES OF COLLAPSIBLE SOIL

Collapsible soils, which are sometimes referred to as *metastable soils,* are unsaturated soils that undergo a large volume change upon saturation. This volume change may or may not be the result of the application of additional load. The behavior of collapsing soils under load is best explained by the typical void ratio–pressure plot (e against log p) for a collapsing soil, as shown in Figure 11.1. Branch ab is determined from the consolidation test on a specimen at its natural moisture content. At a pressure level of p_w, the equilibrium void ratio is e_1. However, if water is introduced into the specimen for saturation, the soil structure will collapse. After saturation, the equilibrium void ratio at the same pressure level p_w is e_2; cd is the branch of e–log p curve under additional load after saturation. Foundations that are constructed on such soils

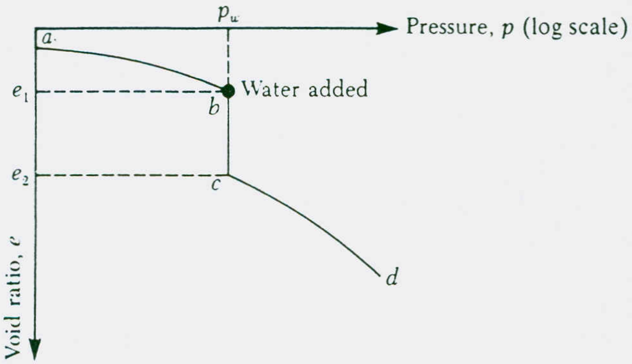


FIGURE 11.1 Nature of variation of void ratio with pressure for a collapsing soil

may undergo large and sudden settlement if and when the soil under them becomes saturated with an unanticipated supply of moisture. This moisture may come from several sources, such as (a) broken water pipelines, (b) leaky sewers, (c) drainage from reservoirs and swimming pools, (d) slow increase of groundwater, and so on. This type of settlement generally causes considerable structural damage. Hence identification of collapsing soils during field exploration is crucial.

The majority of naturally occurring collapsing soils are *aeolian* — that is, wind-deposited sand and/or silts, such as loess, aeolic beaches, and volcanic dust deposits. These deposits have high void ratios and low unit weights and are cohesionless or only slightly cohesive. *Loess* deposits have silt-sized particles. The cohesion in loess may be the result of the presence of clay coatings around the silt-size particles, which holds them in a rather stable condition in an unsaturated state. The cohesion may also be caused by the presence of chemical precipitates leached by rainwater. When the soil becomes saturated, the clay binders lose their strength and hence undergo a structural collapse. In the United States, large parts of the Midwest and arid West have such types of deposit. *Loess* deposits are also found over 15%–20% of Europe and over large parts of China.

Many collapsing soils may be residual soils that are products of weathering of parent rocks. The weathering process produces soils with a large range of particle-size distribution. Soluble and colloidal materials are leached out by weathering, resulting in large void ratios and thus unstable structures. Many parts of South Africa and Zimbabwe have residual soils that are decomposed granites. Sometimes collapsing soil deposits may be left by flash floods and mud flows. These deposits dry out and are poorly consolidated. An excellent review of collapsing soils is that of Clemence and Finbarr (1981).

PHYSICAL PARAMETERS FOR IDENTIFICATION

Several investigators have proposed various methods to evaluate the physical parameters of collapsing soils for identification. Some of these methods are discussed briefly in Table 11.1.

▼ TABLE 11.1 Reported Criteria for Identification of Collapsing Soil[a]

Investigator	Year	Criteria
Denisov	1951	Coefficient of subsidence: $K = \dfrac{\text{void ratio at liquid limit}}{\text{natural void ratio}}$ K = 0.5–0.75: highly collapsible K = 1.0: noncollapsible loam K = 1.5–2.0: noncollapsible soils
Clevenger	1958	If dry unit weight is less than 80 lb/ft^3 (≈12.6 kN/m^3), settlement will be large; if dry unit weight is greater than 90 lb/ft^3 (≈14.1 kN/m^3), settlement will be small.
Priklonski	1952	$K_D = \dfrac{\text{natural moisture content — plastic limit}}{\text{plasticity index}}$ $K_D < 0$: highly collapsible soils $K_D > 0.5$: noncollapsible soils $K_D > 1.0$: swelling soils
Gibbs	1961	Collapse ratio, $R = \dfrac{\text{saturation moisture content}}{\text{liquid limit}}$ This was put into graph form.
Soviet Building Code	1962	$L = \dfrac{e_o - e_L}{1 + e_o}$ where e_o = natural void ratio and e_L = void ratio at liquid limit. For natural degree of saturation less than 60%, if $L > -0.1$, it is a collapsing soil.
Feda	1964	$K_L = \dfrac{w_o}{S_r} - \dfrac{PL}{PI}$ where w_o = natural water content, S_r = natural degree of saturation, PL = plastic limit, and PI = plasticity index. For $S_r < 100\%$, if $K_L > 0.85$, it is a subsident soil.
Benites	1968	A dispersion test in which 2 g of soil are dropped into 12 ml of distilled water and specimen is timed until dispersed; dispersion times of 20 to 30 s were obtained for collapsing Arizona soils.
Handy	1973	Iowa loess with clay (<0.002 mm) contents: <16%: high probability of collapse 16–24%: probability of collapse 24–32%: less than 50% probability of collapse >32%: usually safe from collapse

[a] Modified after Lutenegger and Saber (1988)

Jennings and Knight (1975) suggested a procedure to describe the *collapse potential* of a soil. It can be determined by taking an undisturbed soil specimen at natural moisture content in a consolidation ring. Step loads are applied to the specimen up to a pressure level of 29 lb/in^2 (≈200 kN/m^2). (In Figure 11.1, this is p_w.) At this pressure (p_w = 29 lb/in^2), the specimen is flooded for saturation and left for 24 hours. This test provides the void ratios (e_1 and e_2) before and after flooding. The collapse potential, C_p, may now be calculated as

▼ TABLE 11.2 Relation of Collapse Potential to the Severity of Foundation Problems

C_p (%)	Severity of problem
0–1	No problem
1–5	Moderate trouble
5–10	Trouble
10–20	Severe trouble
20	Very severe trouble

[a] After Clemence and Finbarr (1981)

$$C_p = \Delta\varepsilon = \frac{e_1 - e_2}{1 + e_o} \tag{11.1}$$

where e_o = natural void ratio of the soil
$\Delta\varepsilon$ = vertical strain

The severity of foundation problems associated with a collapsible soil have been correlated with the collapse potential, C_p, by Jennings and Knight (1975). They were summarized by Clemence and Finbarr (1981) and are given in Table 11.2.

Holtz and Hilf (1961) suggested that a loessial soil that has a void ratio large enough to allow its moisture content to exceed its liquid limit upon saturation is susceptible to collapse. So, for collapse

$$w_{(\text{saturated})} \geq LL \tag{11.2}$$

However, for saturated soils

$$e_o = wG_s \tag{11.3}$$

where LL = liquid limit
G_s = specific gravity of soil solids

Combining Eqs. (11.2) and (11.3), for collapsing soils, yields

$$e_o \geq (LL)(G_s) \tag{11.4}$$

The natural dry unit weight, γ_d, of the soil for collapse is

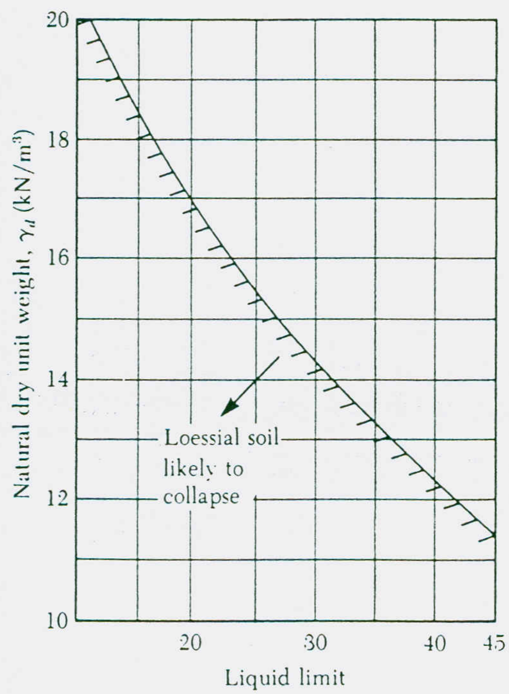
$$\gamma_d \leq \frac{G_s\gamma_w}{1 + e_o} = \frac{G_s\gamma_w}{1 + (LL)(G_s)} \tag{11.5}$$

For an average value of $G_s = 2.65$, the limiting values of γ_d for various liquid limits may now be calculated from Eq. (11.5):

Liquid limit (%)	Limiting values of γ_d (lb/ft³)	(kN/m³)
10	130.8	20.56
15	118.3	18.60
20	108.1	16.99
25	99.5	15.64
30	92.1	14.48
35	85.8	13.49
40	80.3	12.62
45	75.4	11.86

Figure 11.2 shows a plot of the preceding limiting dry unit weights against the corresponding liquid limits. For any soil, if the natural dry unit weight falls below the limiting line, the soil is likely to collapse.

Care should be taken to obtain undisturbed samples for determining the collapse potentials and dry unit weights, preferably block samples cut by hand. The reason is that samples obtained by thin wall tubes may undergo some compression during the sampling process. However, if this procedure is used, the boreholes should be made *without water*.



▼ FIGURE 11.2 Loessial soil likely to collapse

CEDURE FOR CALCULATING LAPSE SETTLEMENT

Jennings and Knight (1975) proposed the following laboratory procedure to determine the collapse settlement of structures upon saturation of soil:

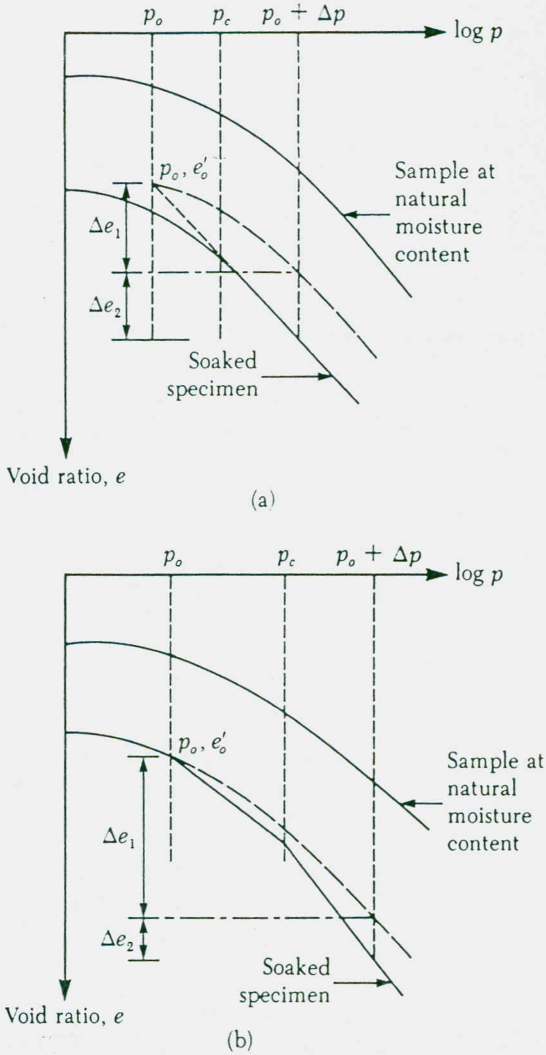
1. Obtain *two* undisturbed soil specimens for tests in a standard consolidation test apparatus (oedometer).
2. Place the two specimens under 0.15 lb/in^2 (1 kN/m^2) pressure for 24 hours.
3. After 24 hours, saturate one specimen by flooding. Keep the other specimen at natural moisture content.
4. After 24 hours of flooding, resume the consolidation test for both specimens by doubling the load (same procedure as the standard consolidation test) to the desired pressure level.
5. Plot the e–log p graphs for both specimens (Figure 11.3a and b).
6. Calculate the *in situ* effective pressure, p_o. Draw a vertical line corresponding to the pressure p_o.
7. From the e–log p curve of the soaked specimen, determine the preconsolidation pressure, p_c. If p_c/p_o = 0.8–1.5, the soil is normally consolidated; however, if $p_c/p_o > 1.5$, it is preconsolidated.
8. Determine e_o', corresponding to p_o from the e–log p curve of the soaked specimen. (This procedure for normally consolidated and overconsolidated soils is shown in Figure 11.3a and b, respectively.)
9. Through point (p_o, e_o') draw a curve that is similar to the e–log p curve obtained from the specimen tested at natural moisture content.
10. Determine the incremental pressure, Δp, on the soil caused by the construction of the foundation. Draw a vertical line corresponding to the pressure of $p_o + \Delta p$ in the e–log p curve.
11. Now, determine Δe_1 and Δe_2. The settlement of soil without change in the natural moisture content is

$$S_1 = \frac{\Delta e_1}{1 + e_o'} (H) \tag{11.6}$$

Also, the settlement caused by collapse in the soil structure is

$$S_2 = \frac{\Delta e_2}{1 + e_o'} (H) \tag{11.7}$$

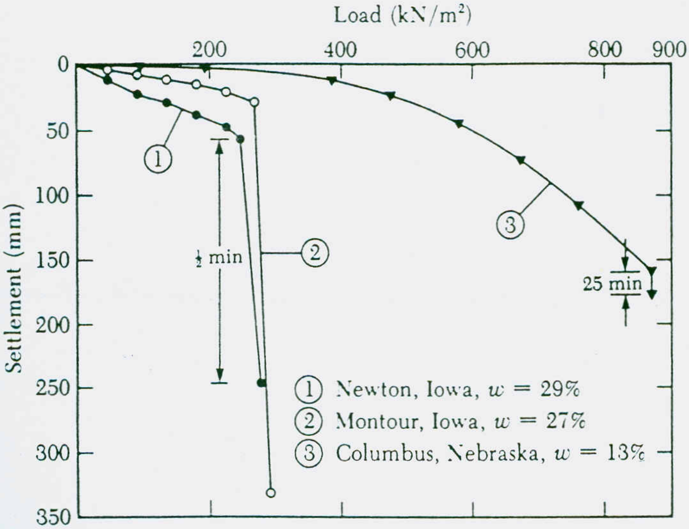
where H = thickness of soil susceptible to collapse



▼ FIGURE 11.3 Settlement calculation from double oedometer test: (a) normally consolidated soil; (b) overconsolidated soil

1.5 FOUNDATION DESIGN IN SOILS NOT SUSCEPTIBLE TO WETTING

For actual foundation design purposes, some standard field load tests may also be conducted. Figure 11.4 shows the results of some field load tests in loess deposits in Nebraska and Iowa. Note that the load–settlement relationships are essentially linear up to a certain critical pressure, p_{cr}, at which there is a breakdown of the soil structure and hence a large settlement. Sudden breakdown of soil structure is more common with soils having a high natural moisture content than with normally dry soils.



▼ **FIGURE 11.4** Results of standard load test on loess deposits in Iowa and Nebraska (adapted from *Foundation Engineering*, Second Edition, by R. B. Peck, W. E. Hanson, and T. H. Thornburn. Copyright 1974 by John Wiley and Sons. Reprinted by permission.)

If enough precautions are taken in the field to prevent moisture from increasing under structures, spread foundations and raft foundations may be built on potentially collapsible soils. However, the foundations must be proportioned so that the critical stresses (Figure 11.4) in the field are never exceeded. A factor of safety of about 2.5 to 3 should be used to calculate the allowable soil pressure, or

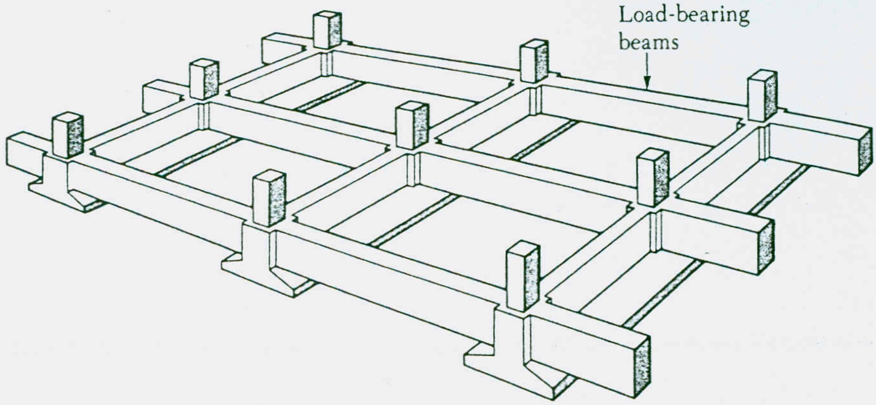
$$\boxed{p_{\text{all}} = \frac{p_{\text{cr}}}{FS}} \tag{11.8}$$

where p_{all} = allowable soil pressure
FS = factor of safety (about 2.5 to 3)

The differential and total settlements of these foundations should be similar to those of foundations designed for sandy soils.

Continuous foundations may be safer than isolated foundations over collapsible soils in that they can effectively minimize differential settlement. Figure 11.5 shows a typical procedure for construction of continuous foundations. This procedure uses footing beams and longitudinal load-bearing beams.

In the construction of heavy structures, such as grain elevators, over collapsible soils, settlements up to about 1 ft ($\approx$0.3 m) are sometimes allowed (Peck, Hanson, and Thornburn, 1974). In this case, tilting of the foundation is not likely to occur because there is no eccentric loading. The total expected settlement for such structures can be estimated from standard consolidation tests on samples of field moisture content. Without eccentric loading, the foundations will exhibit uniform settlement over loessial deposits; however, if the soil is of residual or colluvial nature, settlement



▼ FIGURE 11.5 Continuous foundation with load-bearing beams (after Clemence and Finbarr, 1981)

may not be uniform. The reason is the nonuniformity generally encountered in residual soils.

Extreme caution must be used in building heavy structures over collapsible soils. If large settlements are expected, drilled-shaft and pile foundations should be considered. These types of foundation can transfer the load to a stronger load-bearing stratum.

1.6 FOUNDATION DESIGN IN SOILS SUSCEPTIBLE TO WETTING

If the upper layer of soil is likely to get wet and collapse at some time after construction of the foundation, several design techniques to avoid foundation failure may be considered.

1. If the expected depth of wetting is about 5 to 6.5 ft ($\approx$1.5 to 2 m) from the ground surface, the soil may be moistened and recompacted by heavy rollers. Spread footings and rafts may be constructed over the compacted soil. An alternative to recompaction by heavy rollers is *heavy tamping,* which is sometimes referred to as *dynamic compaction* (see Chapter 12). It consists primarily of dropping a heavy weight repeatedly on the ground. The height of the hammer drop can vary from 25 to 100 ft ($\approx$8 to 30 m). The stress waves generated by the hammer drop help in the densification of the soil.
2. If conditions are favorable, foundation trenches can be flooded with solutions of sodium silicate and calcium chloride to stabilize the soil chemically. The soil will behave like a soft sandstone and resist collapse upon saturation. This method is successful only if the solutions can penetrate to the desired depth; thus it is most applicable to fine sand deposits. Silicates are rather costly and are not generally used. However, in some parts of Denver, silicates have been used very successfully.

The injection of a sodium silicate solution for stabilization of collapsible soil deposits has been used extensively in the former Soviet Union and Bulgaria (Houston and Houston, 1989). This process is used for dry collapsible soils and for wet collapsible soils that are likely to compress under the added weight of the structure to be built and consists of three steps:

Step 1. Injection of carbon dioxide for removal of any water present and preliminary activation of soil

Step 2. Injection of sodium silicate grout

Step 3. Injection of carbon dioxide for neutralization of alkali

3. When the soil layer is susceptible to wetting to a depth of about 10 m, several techniques may be used to cause collapse of the soil *before* foundation construction. Two of these are *vibroflotation* and *ponding* (also called *flooding*). Vibroflotation is used successfully in free-draining soil (see Chapter 12). The procedure of ponding — by constructing low dikes — is utilized at sites that have no impervious layers. However, even after saturation and collapse of the soil by ponding, some additional settlement of the soil may occur after foundation construction. Additional settlement may also be caused by incomplete saturation of the soil at the time of construction. Ponding may be used successfully in the construction of earth dams.
4. If precollapsing of soil is not practical, foundations may be extended beyond the zone of possible wetting, which may require drilled shafts and piles. The design of drilled shafts and piles must take into consideration the effect of negative skin friction resulting from the collapse of the soil structure and the associated settlement of the zone of subsequent wetting.

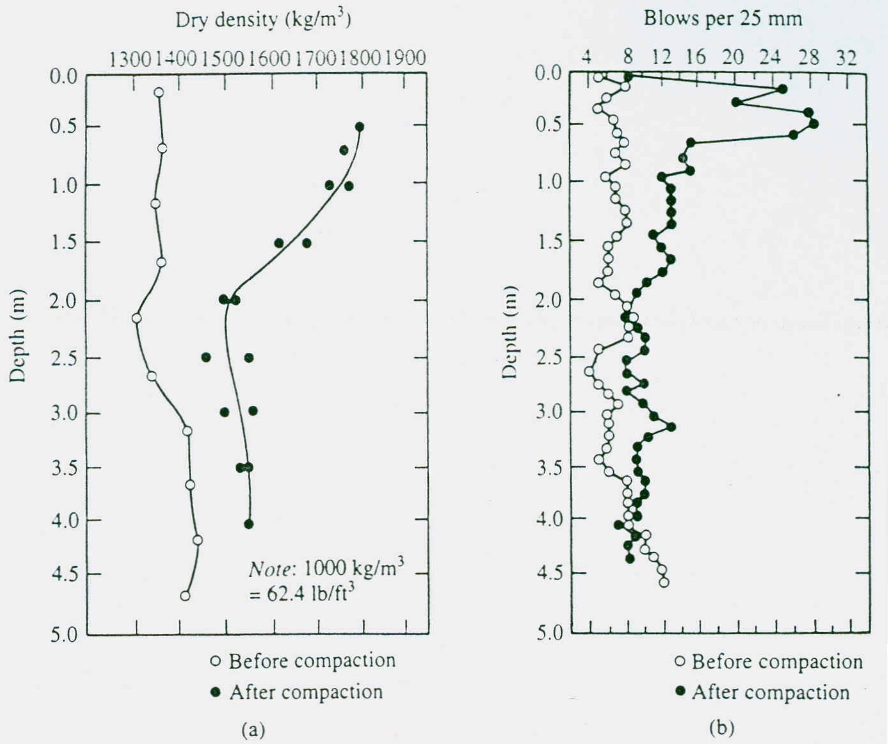
In some cases, a *rock-column type of foundation* (*vibroreplacement*) may also be considered. Rock columns are built with large boulders that penetrate the potentially collapsible soil layer. They act as piles in transferring the load to a more stable soil layer.

SE HISTORIES OF STABILIZATION COLLAPSIBLE SOIL

Use of Dynamic Compaction

Lutenegger (1986) reported the use of dynamic compaction to stabilize a thick layer of friable loess before construction of a foundation in Russe, Bulgaria. During field exploration, the water table was not encountered to a depth of 33 ft (10 m), and the natural moisture content was below the plastic limit. Initial density measurements made on undisturbed soil specimens indicated that the moisture content at saturation would exceed the liquid limit, a property usually encountered in collapsible loess.

For dynamic compaction of the soil, the upper 5.6 ft (1.7 m) of crust material was excavated. A circular concrete weight of 15 ton ($\approx$133 kN) was used as a hammer. At each grid point, compaction was achieved by dropping the hammer 7 to 12 times through a vertical distance of 8.2 ft (2.5 m).



▼ FIGURE 11.6 (a) Dry density before and after compaction; (b) penetration resistance before and after compaction (after Lutenegger, 1986)

Figure 11.6a shows the dry density of the soil before and after compa Figure 11.6b shows the increase in field standard penetration resistance befor after compaction. The increase in dry density of the soil and standard penetr resistance shows that dynamic compaction can be used effectively to stabilize co ible soil.

Chemical Stabilization

Semkin et al. (1986) reported on chemical stabilization of a loessial soil deposit a carbon dioxide, sodium silicate, and carbon dioxide injection scheme (als Houston and Houston, 1989). The site is that of the Interregional Center in Tash which consists of two buildings — one three stories high with strip foundation the other one story high with spread foundations. The loessial soil deposit a site was about 115 ft (35 m) thick, and the water table was at a depth of abo ft (18 m). The natural moisture content and porosity of the soil above the table was 10%–25% and 0.48, respectively.

The Interregional Center was constructed in 1973. Unanticipated leakage conduits in the center caused differential settlement to occur in 1974. Wi shutting down the center, chemical stabilization was used effectively, and the ential settlement was stopped.

'ANSIVE SOILS

EXPANSIVE SOILS—GENERAL

Many plastic clays swell considerably when water is added to them and then shrink with the loss of water. Foundations constructed on these clays are subjected to large uplifting forces caused by the swelling. These forces will induce heaving, cracking, and breakup of both building foundations and slab-on-grade members. Expansive clays cover large parts of the United States, South America, Africa, Australia, and India. In the United States, these clays are predominant in Texas, Oklahoma, and the upper Missouri Valley. In general, potentially expansive clays have liquid limits and plasticity indices greater than about 40 and 15, respectively.

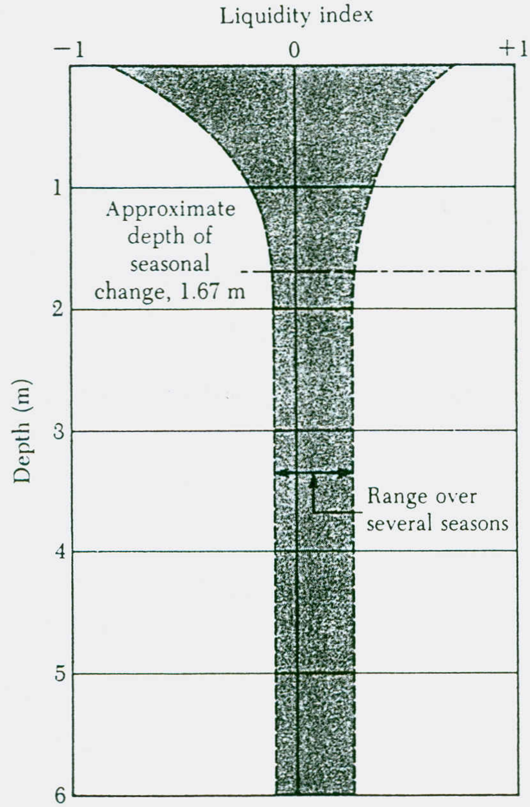
As noted, an increase in moisture content causes clay to swell. The depth in a soil to which periodic changes of moisture occur is usually referred to as the *active zone*. The depth of the active zone varies, depending on location. Some typical active-zone depths in American cities are given in Table 11.3. In some clays and clay shales in the western United States, the depth of the active zone can be as much as 50 ft ($\approx$15 m). The active-zone depth can be easily determined by plotting the liquidity index against the depth of the soil profile over several seasons. Figure 11.7 shows such a plot for the Beaumont formation in the Houston area.

An example of the effect of seasonal change in the active zone related to shrinking and swelling of an expansive soil deposit is shown in Figure 11.8. It is a typical record of vertical ground movement at an open-field test plot in Regina, Saskatchewan (Canada), for the depths below the ground surface indicated. The seasonal ground movement virtually ceases at a depth of about 10–12 ft (3–4 m).

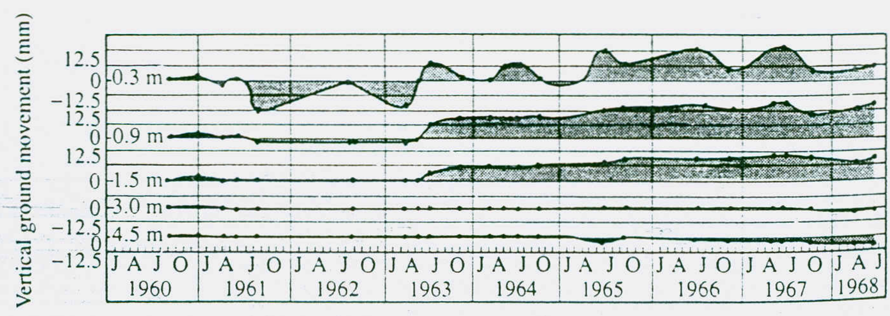
▼ TABLE 11.3 Typical Active-Zone Depths in Some U.S. Cities[a]

City	Depth of active zone (ft)	Depth of active zone (m)
Houston	5 to 10	1.5 to 3
Dallas	7 to 15	2.1 to 4.6
San Antonio	10 to 20	3 to 9
Denver	10 to 15	3 to 4.6

[a] After O'Neill and Poormoayed (1980)



▼ FIGURE 11.7 Active zone in Houston area — Beaumont formation (after O'Neill and Poormoayed, 1980)



▼ FIGURE 11.8 Vertical ground movements for an open-field test plot at Regina, Saskatchewan, as measured by Hamilton 1968 (after Sattler and Fredlund, 1991)

11.9 LABORATORY MEASUREMENT OF SWELL

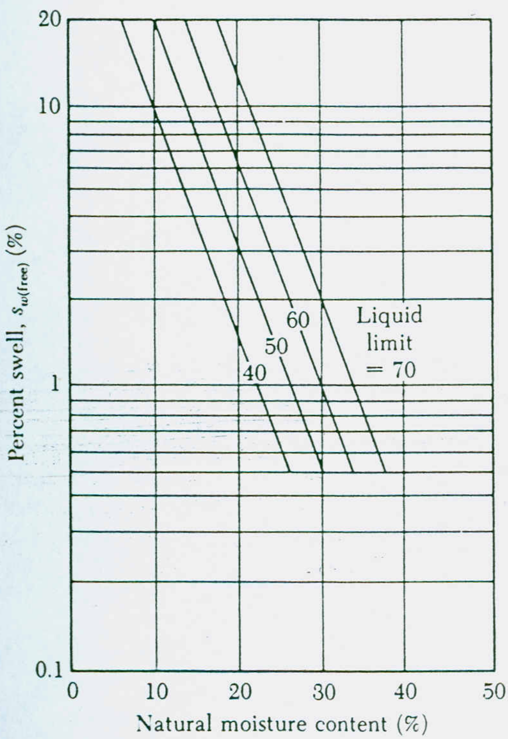
To study the magnitude of possible swell in a clay, simple laboratory oedometer tests can be conducted on undisturbed specimens. Two common tests are the unrestrained swell test and swelling pressure test.

In the *unrestrained swell test,* the specimen is placed in an oedometer under a small surcharge of about 1 lb/in^2 (6.9 kN/m^2). Water is then added to the specimen, and the expansion of the volume of the specimen (that is, height; the area of cross section is constant) is measured until equilibrium is reached. The percent of free swell may be expressed as a ratio:

$$\boxed{s_{w(\text{free})}\ (\%) = \frac{\Delta H}{H}\,(100)} \tag{11.9}$$

where $s_{w(\text{free})}$ = free swell, as a percent
ΔH = height of swell due to saturation
H = original height of the specimen

Vijayvergiya and Ghazzaly (1973) analyzed various soil test results obtained in this manner and prepared a correlation chart of the free swell, liquid limit, and natural moisture content, as shown in Figure 11.9. O'Neill and Poormoayed (1980) developed a relationship for calculating the free surface swell from this chart:



▼ FIGURE 11.9 Relation between percent free swell, liquid limit, and natural moisture content (after Vijayvergiya and Ghazzaly, 1973)

$$\boxed{\Delta S_F = 0.0033 Z s_{w(\text{free})}} \tag{11.10}$$

where ΔS_F = free surface swell
Z = depth of active zone
$s_{w(\text{free})}$ = free swell, as a percent (Figure 11.9)

More recently, Sivapullaiah et al. (1987) suggested a new test method for obtaining a *modified free swell index* for clays, which appears to give a better indication for the swelling potential of clayey soils. This test begins with an oven-dried soil with a mass of about 10 g. The soil mass is well pulverized and transferred into a 100-ml graduated jar containing distilled water. After 24 hours, the swollen sediment volume is measured. The *modified free swell index* is then calculated:

$$\boxed{\text{Modified free swell index} = \frac{V - V_s}{V_s}} \tag{11.11}$$

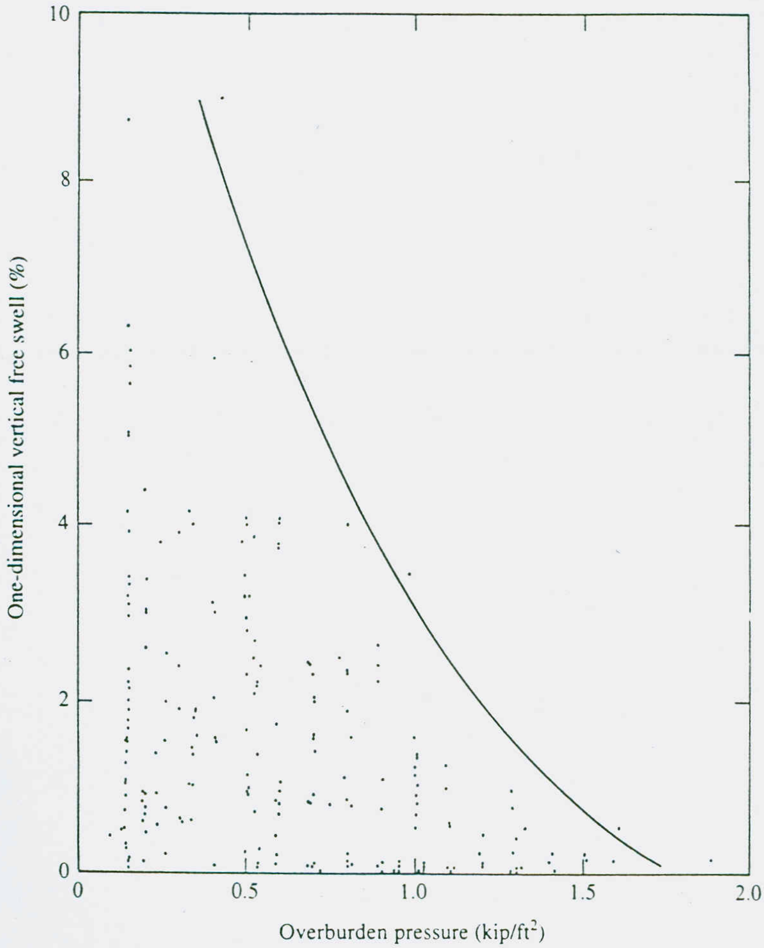
where V = soil volume after swelling
V_s = volume of soil solid $= \dfrac{W_s}{G_s \gamma_w}$
W_s = weight of oven-dried soil
G_s = specific gravity of soil solids
γ_w = unit weight of water

Based on the modified free swell index, the swelling potential of a soil may be qualitatively classified as follows:

Modified free swell index	Swelling potential
<2.5	Negligible
2.5 to 10	Moderate
10 to 20	High
>20	Very high

Sikh (1993) reported the results of several free swell tests on undisturbed soil specimens obtained from Southern California. The tests were conducted by subjecting the soil specimens to the *actual effective overburden pressure*. The results of these tests are given in Figure 11.10. The upper-bound curve indicates that, for an effective over-burden pressure of about 1.4 kip/ft^2 or greater, the vertical free swell [Eq. (11.9)] generally decreases to less than 1%.

The *swelling pressure test* can be conducted by taking a specimen in a consolidation ring and applying a pressure equal to the effective overburden pressure, p_o, plus the approximate anticipated surcharge caused by the foundation, p_s. Water is then added to the specimen. As the specimen starts to swell, pressure is applied in



▼ FIGURE 11.10 One-dimensional vertical free swell of some southern California soils (after Sikh, 1993)

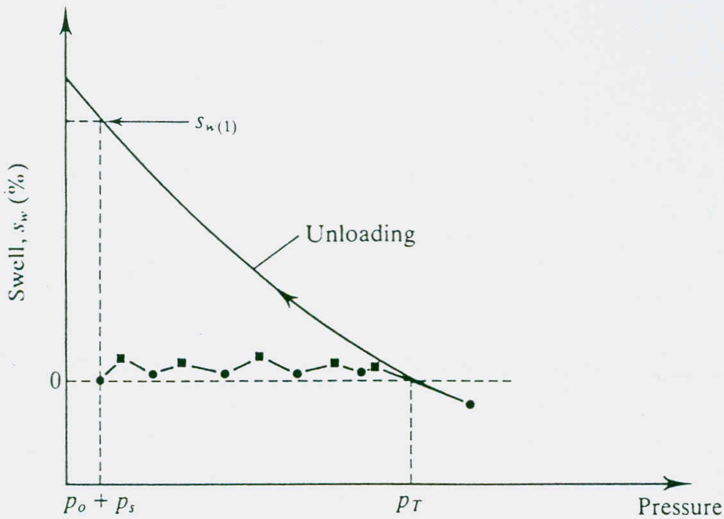
small increments to prevent swelling. It is continued until full swelling pressure is developed on the specimen. At that time, the total pressure is

$$p_T = p_o + p_s + p_1 \tag{11.12}$$

where p_T = total pressure to prevent swelling, or zero swell pressure
p_1 = additional pressure added to prevent swelling after addition of water

Figure 11.11 shows the variation of the percentage of swell against pressure during a swelling pressure test. For more information on this type of test, see Sridharan et al. (1986).

A p_T of about 0.4–0.65 kip/ft^2 (20–30 kN/m^2) is considered to be low, and a p_T of 30–40 kip/ft^2 (1500–2000 kN/m^2) is considered to be high. After zero swell



▼ FIGURE 11.11 Swelling pressure test

pressure is attained, the soil specimen can be unloaded in steps to the level of overburden pressure, p_o. This unloading process will cause the specimen to sv The equilibrium swell for each pressure level is also recorded. The variation of swell, in percent, s_w (%), and the applied pressure on the specimen will be like shown in Figure 11.11.

The *swelling pressure test* can be used to determine the surface heave, ΔS a foundation (O'Neill and Poormoayed, 1980) as

$$\boxed{\Delta S = \sum_{i=1}^{n} [s_{w(1)} (\%)] (H_i) (0.01)} \quad (1$$

where $s_{w(1)}$ (%) = swell, in percent, for layer i under a pressure of $p_o + p_s$ (see Figure 11.11)

ΔH_i = thickness of layer i

▼ EXAMPLE 11.1

A soil profile has an active zone of expansive soil of 2 m. The liquid limit an average natural moisture content during the construction season are 60% and respectively. Determine the free surface swell.

Solution From Figure 11.9 for LL = 60% and w = 30%, $s_{w(\text{free})}$ = 1%. From Eq. (1

$$\Delta S_F = 0.0033 Z s_{w(\text{free})}$$

Hence

$$\Delta S_F = 0.0033(2)(1)(1000) = \mathbf{6.6\ mm}$$

PLE 11.2

An expansive soil profile has an active-zone thickness of 5.2 m. A shallow foundation is to be constructed 1.2 m below the ground surface. A swelling pressure test provided the following data:

Depth below ground surface (m)	Swell under overburden and estimated foundation surcharge pressure, $s_{w(1)}$ (%)
1.2	3.0
2.2	2.0
3.2	1.2
4.2	0.55
5.2	0.0

a. Estimate the total possible swell under the foundation.
b. If the allowable total swell is 15 mm, what would be the necessary undercut?

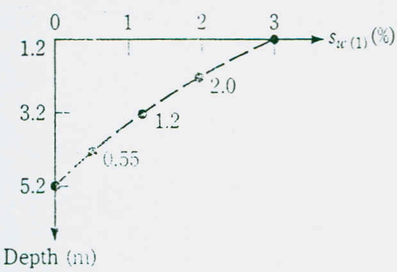
Solution

Part a:

Figure 11.12 shows the plot of depth versus $s_{w(1)}$ (%). The area of this diagram will be the total swell. Thus

$$\Delta S = \frac{1}{100}\left[\left(\frac{1}{2}\right)(0.55 + 0)(1) + \left(\frac{1}{2}\right)(0.55 + 1.2)(1) + \left(\frac{1}{2}\right)(1.2 + 2)(1) + \left(\frac{1}{2}\right)(2 + 3)(1)\right]$$

$$= 0.0525 \text{ m} = \mathbf{52.5\ mm}$$



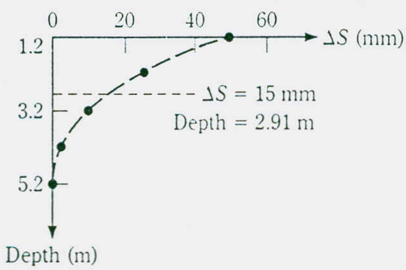
▼ FIGURE 11.12

Part b:

Total swell at various depths can be calculated as follows:

Depth (m)	Total swell, ΔS (m)
5.2	0
4.2	$0 + \frac{1}{2}(0.55 + 0)(1)(1/100) = 0.00275$
3.2	$0.00275 + \frac{1}{2}(1.2 + 0.55)(1)(1/100) = 0.0115$
2.2	$0.0115 + \frac{1}{2}(2 + 1.2)(1)(1/100) = 0.0275$
1.2	$0.0275 + \frac{1}{2}(2 + 3)(1)(1/100) = 0.0525$

The plot of ΔS versus depth is shown in Figure 11.13. From this figure, the depth of undercut is $2.91 - 1.2 =$ **1.71 m below the bottom of the foundation.**



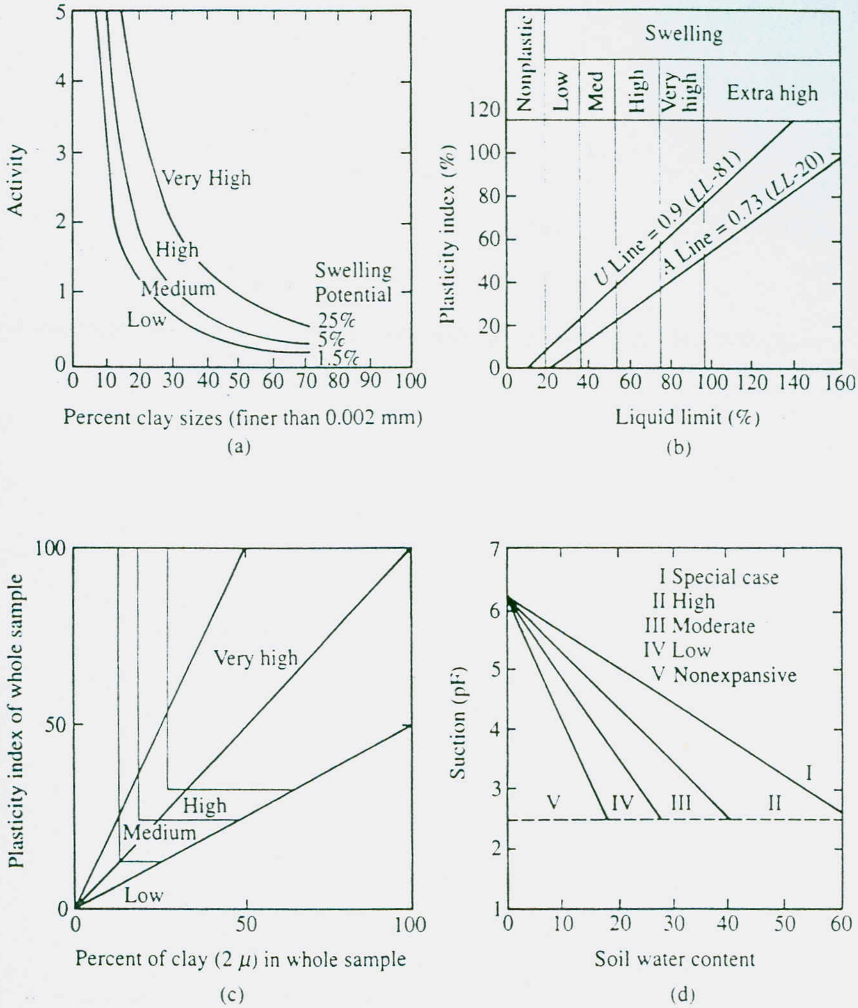
▼ FIGURE 11.13 ▲

11.10 CLASSIFICATION OF EXPANSIVE SOIL BASED ON INDEX TESTS

Classification systems for expansive soils are based on the problems they create in the construction of foundations (potential swell). Most of the classifications contained in the literature are summarized in Figure 11.14 and Table 11.4. However, the classification system developed by U.S. Army Waterways Experiment Station (Snethen et al., 1977) is the one most widely used in the United States. It has also been summarized by O'Neill and Poormoayed (1980); see Table 11.5 (p. 749).

11.11 FOUNDATION CONSIDERATIONS FOR EXPANSIVE SOILS

If a soil has a low swell potential, standard construction practices may be followed. However, if the soil possesses a marginal or high swell potential, precautions need to be taken, which may entail



▼ **FIGURE 11.14** Commonly used criteria for determining swell potential (after Abduljauwad and Al-Sulaimani, 1993)

1. Replacing the expansive soil under the foundation
2. Changing the nature of the expansive soil by compaction control, prewetting, installation of moisture barriers, and/or chemical stabilization
3. Strengthening the structures to withstand heave, constructing structures that are flexible enough to withstand the differential soil heave without failure, or constructing isolated deep foundations below the depth of the active zone

One particular method may not be sufficient in all situations. Combining several techniques may be necessary, and local construction experience should always be

TABLE 11.4 Summary of Some Criteria for Identifying Swell Potential (after Abduljauwad and Al-Sulaimani, 1993)

Reference	Criteria	Remarks
Holtz (1959)	$CC > 28$, $PI > 35$, and $SL < 11$ (very high) $20 \leq CC \leq 31$, $25 \leq PI \leq 41$, and $7 \leq SL \leq 12$ (high) $13 \leq CC \leq 23$, $15 \leq PI \leq 28$, and $10 \leq SL \leq 16$ (medium) $CC \leq 15$, $PI \leq 18$, and $SL \geq 15$ (low)	Based on *CC*, *PI*, and *SL*
Seed et al. (1962)	See Figure 11.14a	Based on oedometer test using compacted specimen, percentage of clay < 2 μm and activity
Altmeyer (1955)	$LS < 5$, $SL > 12$, and $PS < 0.5$ (noncritical) $5 \leq LS \leq 8$, $10 \leq SL \leq 12$, and $0.5 \leq PS \leq 1.5$ (marginal) $LS > 8$, $SL < 10$, and $PS > 1.5$ (critical)	Based on *LS*, *SL*, and *PS* Remolded sample ($\rho_{d(\max)}$ and w_{opt}) Soaked under 6.9 kPa surcharge
Dakshanamanthy and Raman (1973)	See Figure 11.14b	Based on plasticity chart
Raman (1967)	$PI > 32$ and $SI > 40$ (very high) $23 \leq PI \leq 32$ and $30 \leq SI \leq 40$ (high) $12 \leq PI \leq 23$ and $15 \leq SI \leq 30$ (medium) $PI < 12$ and $SI < 15$ (low)	Based on *PI* and *SI*
Sowers and Sowers (1970)	$SL < 10$ and $PI > 30$ (high) $10 \leq SL \leq 12$ and $15 \leq PI \leq 30$ (moderate) $SL > 12$ and $PI < 15$ (low)	Little swell will occur when w_0 results in *LI* of 0.25
Van Der Merwe (1964)	See Figure 11.14c	Based on *PI*, percentage of clay <2 μm, and activity
Uniform Building Code, 1968	$EI > 130$ (very high) and $91 \leq EI \leq 130$ (high) $51 \leq EI \leq 90$ (medium) and $21 \leq EI \leq 50$ (low) $0 \leq EI \leq 20$ (very low)	Based on oedometer test on compacted specimen with degree of saturation close to 50% and a surcharge of 6.9 kPa
Snethen (1984)	$LL > 60$, $PI > 35$, $\tau_{nat} > 4$, and $SP > 1.5$ (high) $30 \leq LL \leq 60$, $25 \leq PI \leq 35$, $1.5 \leq \tau_{nat} \leq 4$, and $0.5 \leq SP \leq 1.5$ (medium) $LL < 30$, $PI < 25$, $\tau_{nat} < 1.5$, and $SP < 0.5$ (low)	*PS* is representative for field condition can be used without τ_{nat}, but accuracy will be reduced
Chen (1988)	$PI \geq 35$ (very high) and $20 \leq PI \leq 55$ (high) $10 \leq PI \leq 35$ (medium) and $PI \leq 15$ (low)	Based on *PI*
McKeen (1992)	Figure 11.14d	Based on measurements of soft water content, suction, and volume change on drying
Vijayvergiya and Ghazzaly (1973)	$\log SP = (1/12)(0.44LL - w_o + 5.5)$	Empirical equations
Nayak and Christensen (1974)	$SP = (0.00229PI)(1.45C)/w_0 + 6.38$	Empirical equations
Weston (1980)	$SP = 0.00411(LL_w)^{4.17} q^{-3.86} w_0^{-2.33}$	Empirical equations

Note: C = clay, %
CC = colloidal content, %
EI = Expansion index = 100 × percent swell × fraction passing no. 4 sieve
LI = liquidity index, %
LL = liquid limit, %
LL_w = weighted liquid limit, %
LS = linear shrinkage, %
PI = plasticity index, %
PS = probable swell, %
q = surcharge
SI = shrinkage index = $LL - SL$, %
SL = shrinkage limit, %
SP = swell potential, %
w_0 = natural soil moisture
w_{opt} = optimum moisture content, %
τ_{nat} = natural soil suction in tsf
$\rho_{d(\max)}$ = max dry density

TABLE 11.5 Expansive Soil Classification System[a]

Liquid limit	Plasticity index	Potential swell (%)	Potential swell classification
<50	<25	<0.5	Low
50–60	25–35	0.5–1.5	Marginal
>60	>35	>1.5	High

Potential swell = vertical swell under a pressure equal to overburden pressure

[a] Compiled from O'Neill and Poormoayed (1980)

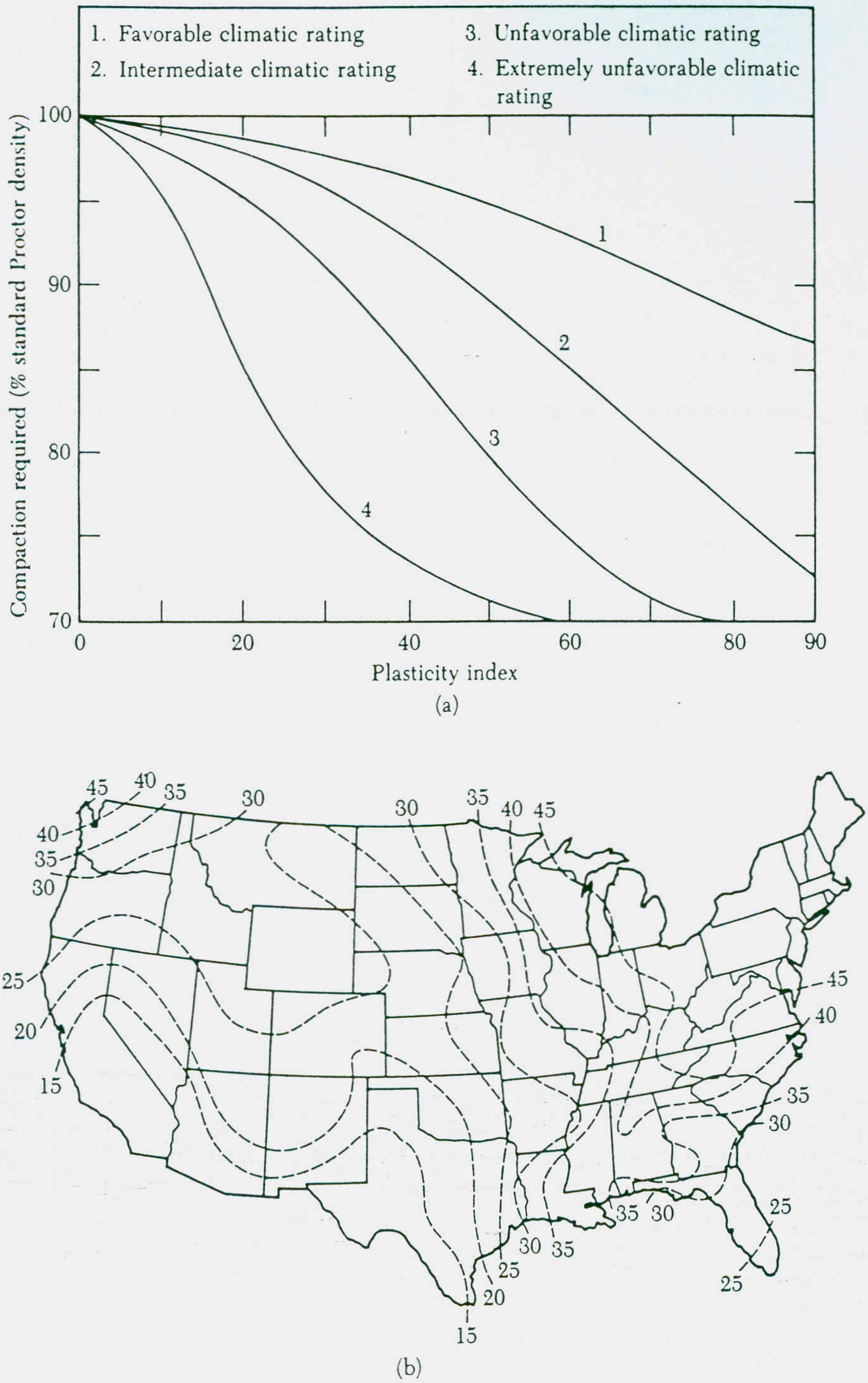
considered. Following are details of some of the commonly used techniques of dealing with expansive soils.

Replacement of Expansive Soil

When shallow, moderately expansive soils are present at the surface, they can be removed and replaced by less expansive soils and then compacted properly.

Changing the Nature of Expansive Soil

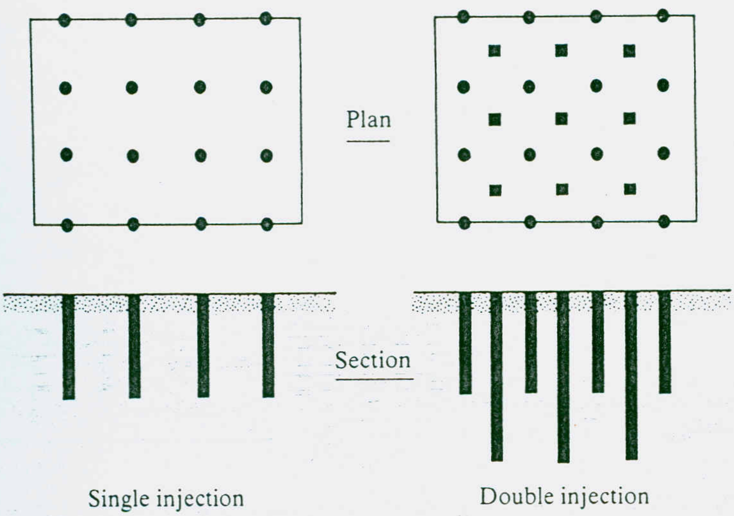
1. *Compaction*: Heave of expansive soils decreases substantially when the soil is compacted to a lower unit weight on the high side of the optimum moisture content (possibly 3–4% above the optimum moisture content). Even under such conditions, a slab-on-ground type of construction should not be considered when the total probable heave is expected to be about 1.5 in. (38 mm) or more. Figure 11.15 shows the recommended limits of soil compaction in the field for reduction of heave. Note that the recommended dry unit weights are based on climatic ratings. According to U.S. Weather Bureau data, a climatic rating of 15 represents an extremely unfavorable climatic condition; a rating of 45 represents a favorable climatic condition. The isobars of climatic rating for the continental United States are shown in Figure 11.15b.
2. *Prewetting*: One technique for increasing the moisture content of the soil is by ponding and hence achieving most of the heave before construction. However, this technique may be time-consuming because the seepage of water through highly plastic clays is slow. After ponding, 4–5% of hydrated lime may be added to the top layer of the soil to make it less plastic and more workable (Gromko, 1974).
3. *Installation of moisture barriers*: The long-term effect of the differential heave can be reduced by controlling the moisture variation in the soil. It is achieved by providing vertical moisture barriers about 5 ft (≈1.5 m) deep around the perimeter of slabs for the slab-on-grade type of construction. These moisture



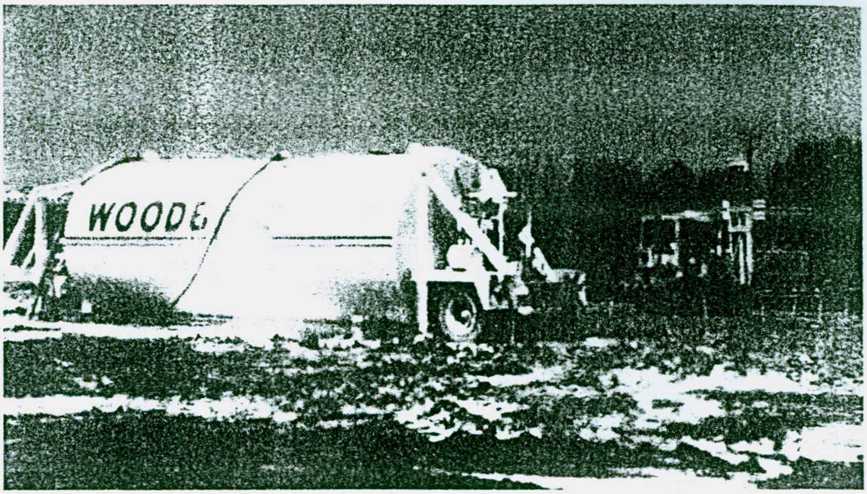
▼ FIGURE 11.15 (a) Soil compaction requirement based on climatic rating; (b) equivalent climatic rating of the United States (after Gromko, 1974) barriers may be constructed in trenches filled with gravel, lean concrete, or impervious membranes.

4. *Stabilization of soil*: Chemical stabilization with the aid of lime and cement has often proved useful. A mix containing about 5% lime is sufficient in most cases. Lime or cement and water are mixed with the top layer of soil and compacted. The addition of lime or cement will decrease the liquid limit, the plasticity index, and the swell characteristics of the soil. This type of stabilization work can be done to a depth of 3–5 ft (≈1–1.5 m). Hydrated high-calcium lime and dolomite lime are generally used for lime stabilization.

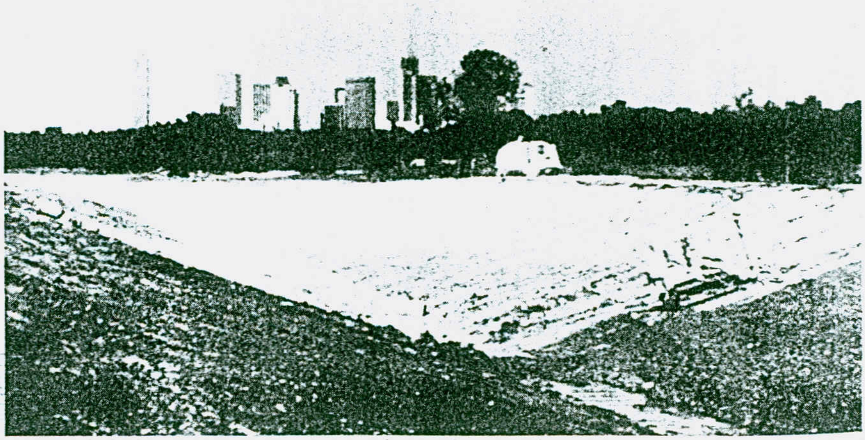
Another method of stabilization of expansive soil is the *pressure injection* of lime slurry or lime–fly ash slurry into the soil, usually to a depth of 12–16 ft (4–5 m) and occasionally deeper to cover the active zone. Further details of the pressure injection technique are presented in Chapter 12. Depending on the soil conditions at a site, single or multiple injections can be planned, as shown in Figure 11.16. Figure 11.17 shows the slurry pressure injection work for a building pad. The stakes marked are the planned injection points. Figure 11.18 shows lime–fly ash stabilization by pressure injection of the bank of a canal that had experienced sloughs and slides.



▼ **FIGURE 11.16** Multiple lime slurry injection planning for a building pad



▼ FIGURE 11.17 Pressure injection of lime slurry for a building pad (Courtesy of GKN Hayward Baker, Inc., Woodbine Division, Ft. Worth, Texas)



▼ FIGURE 11.18 Slope stabilization of a canal bank by pressure injection of lime–fly ash slurry (Courtesy of GKN Hayward Baker, Inc., Woodbine Division, Ft. Worth, Texas)

11.12 CONSTRUCTION ON EXPANSIVE SOILS

Care must be exercised in choosing the type of foundation to be used on expansi soils. Table 11.6 shows some recommended construction procedures based on t total predicted heave, ΔS, and the length-to-height ratio of the wall panels.

1.6 Construction Procedures for Expansive Clay Soils[a]

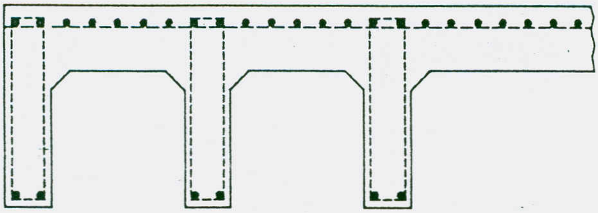
cted heave (mm)		Recommended construction	Method	Remarks
5	$L/H = 2.5$			
	12.7	No precaution		
	12.7 to 50.8	Rigid building tolerating movement (steel reinforcement as necessary)	*Foundations*: Pads, Strip footings, Raft (waffle)	Footings should be small and deep, consistent with the soil-bearing capacity. Rafts should resist bending.
			Floor slabs: Waffle, Tile	Slabs should be designed to resist bending and should be independent of grade beams.
			Walls:	Walls on a raft should be as flexible as the raft. No rigid connections vertically. Brick works should be strengthened with tie bars or bands.
	50.8 to 101.6	Building damping movement	*Joints*: Clear, Flexible	Contacts between structural units should be avoided; or flexible, waterproof material may be inserted in the joints.
			Walls: Flexible, Unit construction, Steel frame	Walls or rectangular building units should heave as a unit.
			Foundations: Three point, Cellular, Jacks	Cellular foundations allow slight soil expansion to reduce swelling pressure. Adjustable jacks can be inconvenient to owners. Three-point loading allows motion without duress.
	>101.6	Building independent of movement	*Foundation drilled shaft*: Straight shaft, Bell bottom	Smallest-diameter and widely spaced piers compatible with load should be placed. Clearance should be allowed under grade beams.
			Suspended floor:	Floor should be suspended on grade beams 305 to 460 mm above the soil.

ko, 1974

For example, Table 11.6 proposes the use of waffle slabs as an alternative in designing rigid buildings capable of tolerating movement. Figure 11.19 shows a schematic diagram of a waffle slab. In this type of construction, the ribs hold the structural load. The waffle voids allow the expansion of soil.

Table 11.6 also suggests the use of drilled shaft foundation with a suspended floor slab for the construction of structures independent of movement. Figure 11.20a shows a schematic diagram of such an arrangement. The bottom of the shafts should be placed below the active zone of the expansive soil. For the design of the shafts, the uplifting force, U, may be estimated (Figure 11.20b) from the equation

$$U = \pi D_s Z p_T \tan \phi_{ps} \tag{11.14}$$



▼ FIGURE 11.19 Waffle slab

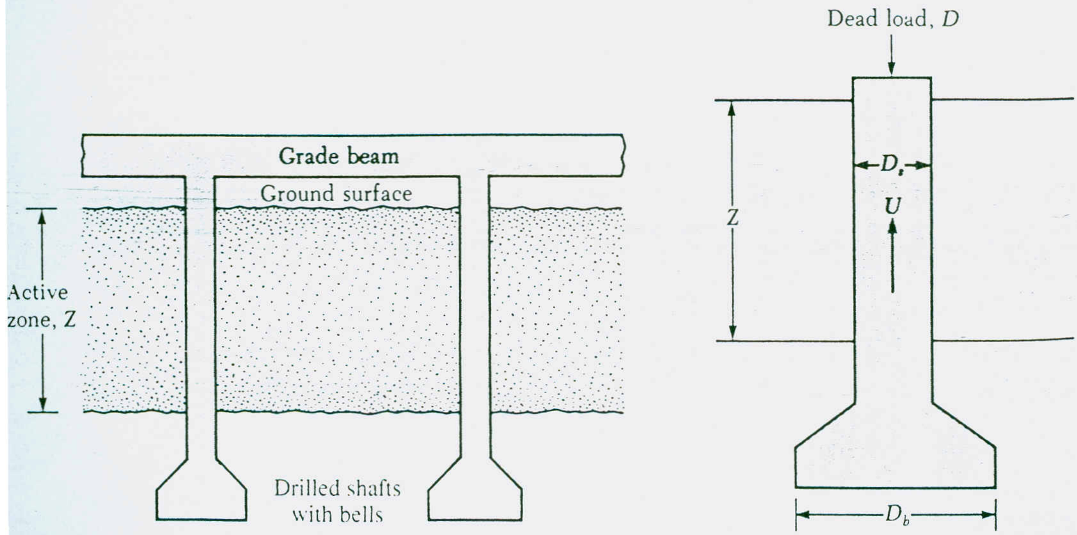
where D_s = diameter of the shaft
Z = depth of the active zone
ϕ_{ps} = effective angle of plinth–soil friction
p_T = pressure for zero horizontal swell (see Figure 11.11; $p_T = p_o + p_s + p_1$)

In most cases, the value of ϕ_{ps} varies between 10° and 20°. An average value of th zero horizontal swell pressure must be determined in the laboratory. In the absenc of laboratory results, $p_T \tan \phi_{ps}$ may be considered equal to the undrained shea strength of clay, c_u, in the active zone.

The belled portion of the drilled shaft will act as an anchor to resist the upliftin force. Ignoring the weight of the drilled shaft

$$Q_{net} = U - D \quad (11.15$$

where Q_{net} = net uplift load
D = dead load



▼ FIGURE 11.20 (a) Construction of drilled shafts with bells and grade beam; (b) definition of parameters in Eq. (11.14)

Now

$$Q_{\text{net}} \approx \frac{c_u N_c}{FS}\left(\frac{\pi}{4}\right)(D_b^2 - D_s^2) \tag{11.16}$$

where c_u = undrained cohesion of the clay in which the bell is located

Combining Eqs. (11.15) and (11.16) gives

$$\boxed{U - D = \frac{c_u N_c}{FS}\left(\frac{\pi}{4}\right)(D_b^2 - D_s^2)} \tag{11.17}$$

where N_c = bearing capacity factor
FS = factor of safety
D_b = diameter of the bell of the drilled shaft

Conservatively, N_c (Tables 3.4 and 3.5) is

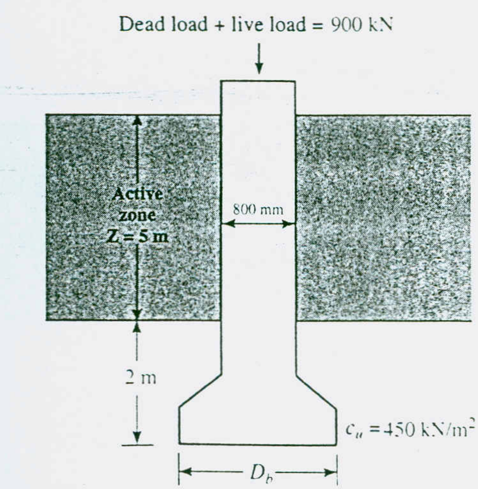
$$N_c \approx N_{c(\text{strip})} F_{cs} = N_{c(\text{strip})}\left(1 + \frac{N_q B}{N_c L}\right) \approx 5.14\left(1 + \frac{1}{5.14}\right) = 6.14$$

An example of a drilled-shaft design is given in Example 11.3.

PLE 11.3

Figure 11.21 shows a drilled shaft with a bell. The depth of the active zone is 5 m. The zero swell pressure of the swelling clay (p_T) is 450 kN/m². For the drilled shaft, the dead load (D) is 600 kN and the live load is 300 kN. Assume ϕ_{ps} = 12°.

a. Determine the diameter of the bell, D_b.
b. Check the bearing capacity of the drilled shaft assuming zero uplift force.



▼ FIGURE 11.21

Solution

Part a: Determining the Bell Diameter, D_b

The uplift force, Eq. (11.14), is

$$U = \pi D_s Z p_T \tan \phi_{ps}$$

Given: $Z = 5$ m and $p_T = 450$ kN/m². Then

$$U = \pi (0.8)(5)(450) \tan 12° = 1202 \text{ kN}$$

Assume the dead load and live load to be zero, and FS in Eq. (11.17) to be 1.25. So, from Eq. (11.17),

$$U = \frac{c_u N_c}{FS}\left(\frac{\pi}{4}\right)(D_b^2 - D_s^2)$$

$$1202 = \frac{(450)(6.14)}{1.25}\left(\frac{\pi}{4}\right)(D_b^2 - 0.8^2); \qquad D_b = \mathbf{1.15\ m}$$

The factor of safety against uplift with the dead load also should be checked. A factor of safety of at least 2 is desirable. So, from Eq. (11.17)

$$FS = \frac{c_u N_c \left(\frac{\pi}{4}\right)(D_b^2 - D_s^2)}{U - D}$$

$$= \frac{(450)(6.14)\left(\frac{\pi}{4}\right)(1.15^2 - 0.8^2)}{1202 - 600} = \mathbf{2.46 > 2\text{—OK}}$$

Part b: Check for Bearing Capacity

Assume that $U = 0$. Then

$$\text{Dead load} + \text{live load} = 600 + 300 = 900 \text{ kN}$$

$$\text{Downward load per unit area} = \frac{900}{\left(\frac{\pi}{4}\right)(D_b^2)} = \frac{900}{\left(\frac{\pi}{4}\right)(1.15)^2} = 866.5 \text{ kN/m}^2$$

$$\text{Net bearing capacity of the soil under the bell} = q_{u(\text{net})} = c_u N_c = (450)(6.14) = 2763 \text{ kN/m}^2$$

Hence the factor of safety against bearing capacity failure is

$$FS = \frac{2763}{866.5} = \mathbf{3.19 > 3\text{—OK}}$$

▲