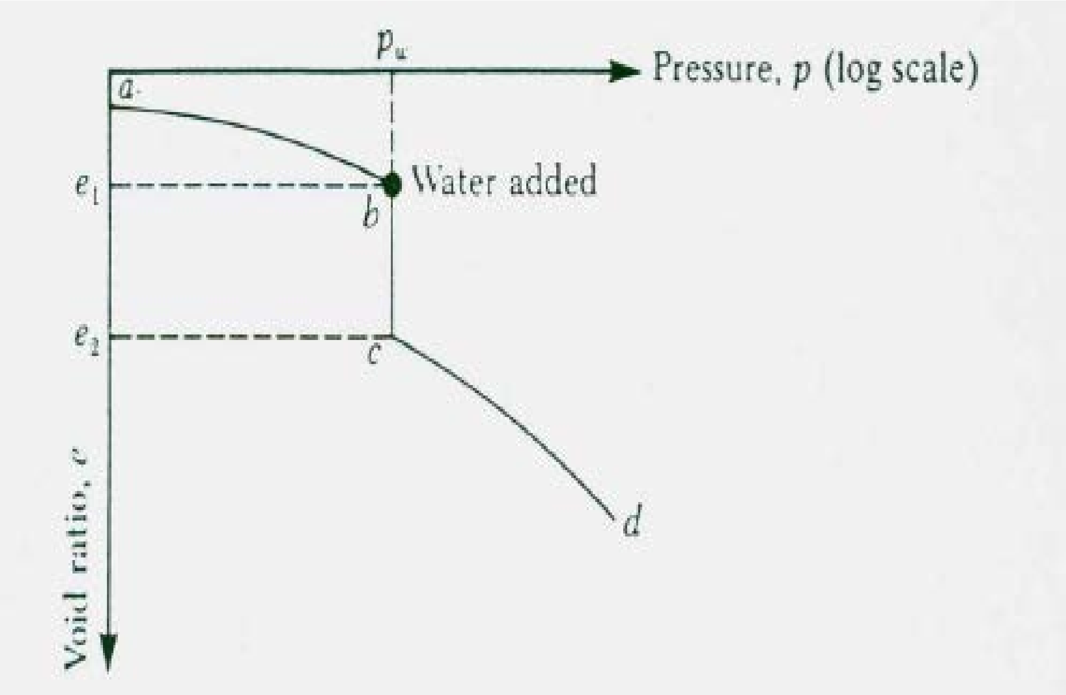
FOUNDATIONS ON DIFFICULT SOILS

Difficult Soils

- Collapsible Soils
- Expansive Soils

Collapsible Soils

Collapsible Soils: 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.



At a pressure level Pu, the equilibrium void ratio is e1. However, if water is introduced into the specimen for saturation, the soil structure will collapse.

The foundations that are constructed on such soils may undergo large and sudden settlement if and when the soil becomes unanticipated supply of moisture.

Collapsible Soils

The moisture can come from:

- Broken water pipelines
- Leaky sewers
- Drainage from reservoirs and swimming pools
- Slow increase of ground water

and so on.

Collapsible Soils

Collapsible soils have

- High void ratios
- Low unit weights (10 and 17 kN/m3)
- Cohesionless or slighlty cohesive
- Typical collapsible soils are lightly colored
- low in plasticity with LL < 45, PI < 25
- 60 to 40 percent porosity (Collapse rarely occurs in soil with porosity less than 40 percent.)

Potential Collapse. The potential for collapse should be determined from results of aconsolidometer test. The soil may then be modified as needed using soil improvement methods to reduce or eliminate the potential for collapse.

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

Investigator	Year	Criteria
Denisov	1951	Coefficient of subsidence: $K = \frac{\text{void ratio at liquid limit}}{\text{natural void ratio}}$ 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/ft³ (≈12.6 kN/m³), settlement will be large; if dry unit weight is greater than 90 lb/ft³ (≈14.1 kN/m³), settlement will be small.
Priklonski	1952	$K_D = \frac{\text{natural moisture content — plastic limit}}{\text{plasticity index}}$ $K_D < 0$: highly collapsible soils $K_D > 0.5$: noncollapsible soils $K_D > 1.0$: swelling soils
Gibbs	1961	Collapse ratio, $R = \frac{\text{saturation moisture content}}{\text{liquid limit}}$ This was put into graph form.
Soviet Building Code	1962	$L = \frac{e_o - e_L}{1 + e_o}$ 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	$K_L = \frac{w_o}{S_r} - \frac{PL}{PI}$ 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)

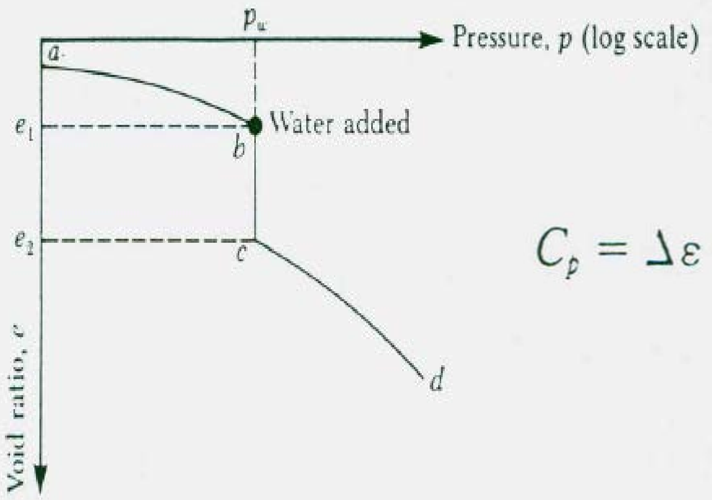
Jennings and Knight (1975)

Take an undisturbed soil specimen at natural moisture content

Consolidation test

Apply step loads up to 200 kN/m2

Saturate the sample (left for 24 hours)



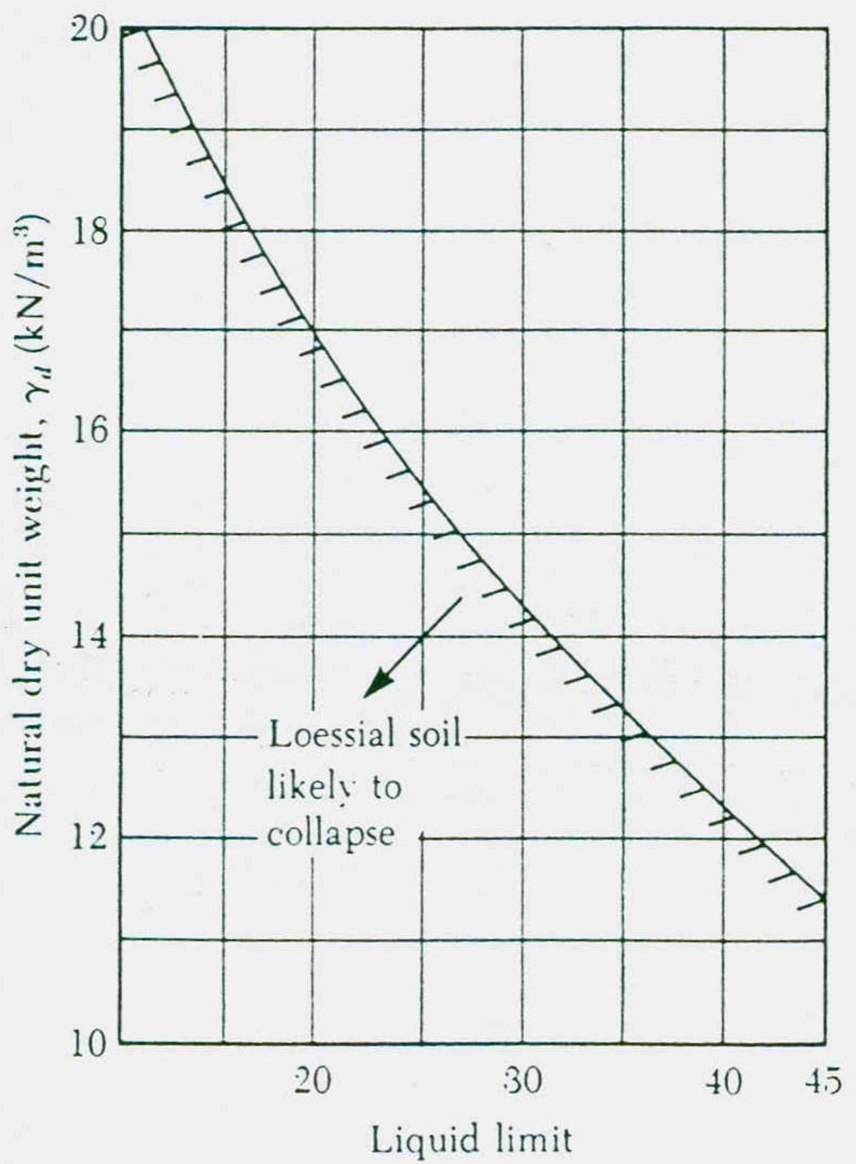
$$C_p = \Delta\varepsilon = \frac{e_1 - e_2}{1 + e_o}$$

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)

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

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



Collapse Settlement

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

$$S_1 = \frac{\Delta e_1}{1 + e'_o}(H) \qquad (11.6)$$

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

$$S_2 = \frac{\Delta e_2}{1 + e'_o}(H) \qquad (11.7)$$

where H = thickness of soil susceptible to collapse

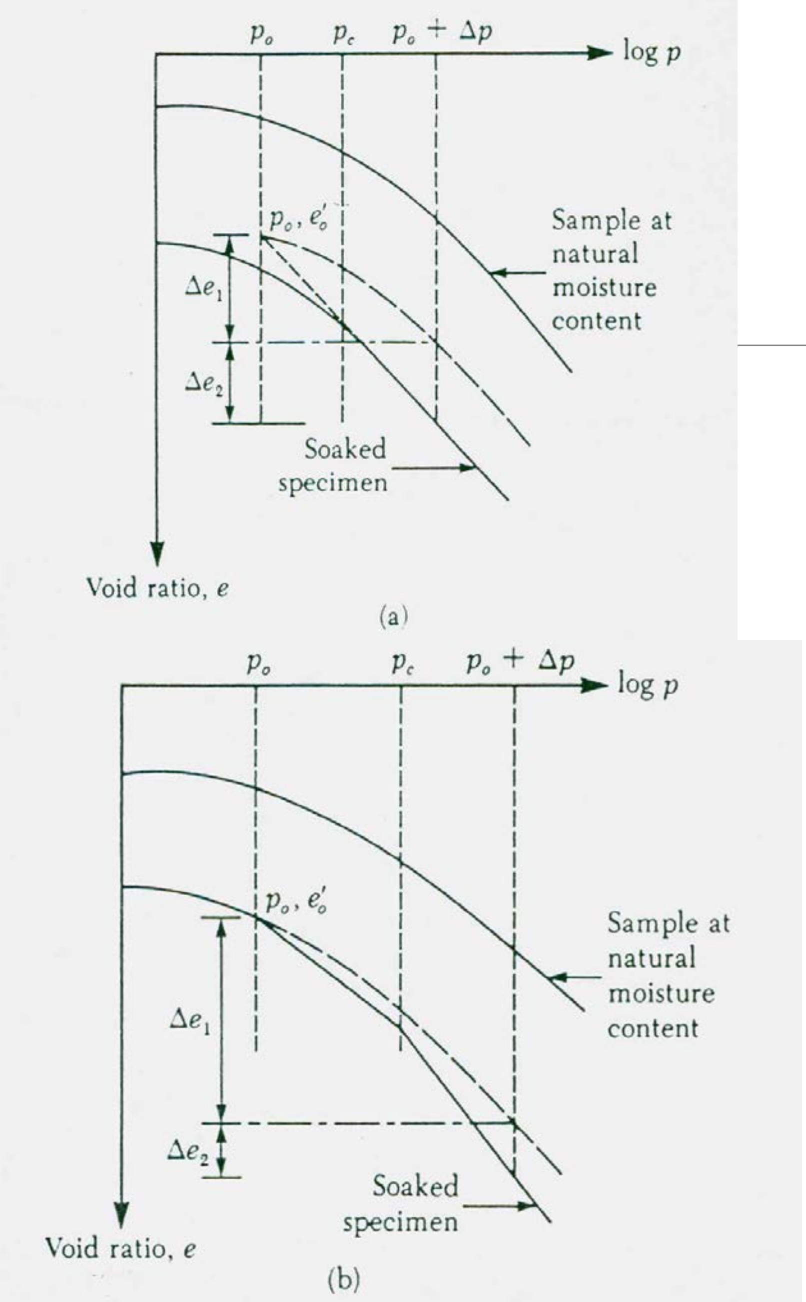


FIGURE 11.3 Settlement calculation from double oedometer test: (a) normally consolidated soil; (b) overconsolidated soil

Stabilization of Collapsible Soil

Dynamic Compaction

Chemical Stabilization (carbon dioxide, sodium silicate)

Expansive Soils

Expansive soils are soils that expand when water is added, and shrink when they dry out. They usually contain some form of expansive clay mineral, such as smectite or vermiculite, that are able to absorb water and swell, increasing in volume, when they get wet and shrink when they dry.

The more water they absorb, the more their volume increases. Expansions of 10% or more are common. This change in volume can exert enough force on a building or other structure to cause damage.

Cracked foundations, floors, and basement walls are typical types of damage done by swelling soils. Damage to the upper floors of the building can occur when motion in the structure is significant.

Expansive Soils

The amount by which the ground can shrink or swell is determined by the water content in the near-surface (active) zone; significant activity usually occurs to about 3 m depth, unless this zone is extended by the presence of tree roots (Driscoll 1983; Biddle 1998, 2001).

During rainfall these soils can absorb large quantities of water becoming sticky and heavy and causing heave, or lifting, of structures, and during prolonged periods of drought they can become very hard, causing shrinkage of the ground and differential settlement.

This hardening and softening is known as “shrink–swell” behavior and presents a significant geotechnical and structural challenge to anyone wishing to build on, or in, them.

The main factors controlling this behavior are the clay content and mineralogy, the in-situ effective stresses, and the stiffness of the material. Aspects such as original geological environment, climate, topography, land-use, and weathering affect these factors, and hence shrink–swell susceptibility.

Expansive Soils

Expansive soils are found throughout many regions of the world, particularly in arid and semiarid regions, as well as where wet conditions occur after prolonged periods of drought.

Expansive soils can cause heaving of structures when they swell and differential settlement when they shrink. Damage to a structure is possible when as little as 3% volume expansion takes place (Jones 2002), especially where these changes are distributed unevenly beneath the foundations

Structural damage to house caused by “end lift” (by Peter Kelsey & Partners)

Expansive Soils

The shrink–swell potential of expansive soils is determined by its initial water content; void ratio; internal structure and vertical stresses; as well as the type and amount of clay minerals in the soil (Bell and Culshaw 2001).

These minerals determine the natural expansiveness of the soil, and include smectite, montmorillonite, nontronite, vermiculite, illite, and chlorite.

Generally, the larger the amounts of these minerals present in the soil, the greater the expansive potential.

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

Expansive Soils

Unrestrained swell test:

Specimen is placed under a surcharge of 6.9 kN/m2.

Water is added to the specimen

Expansion is measured (height, the cross sectional area is constant)

$$S_{u(\text{free})}\ (\%) = \frac{\Delta H}{H}(100)$$

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

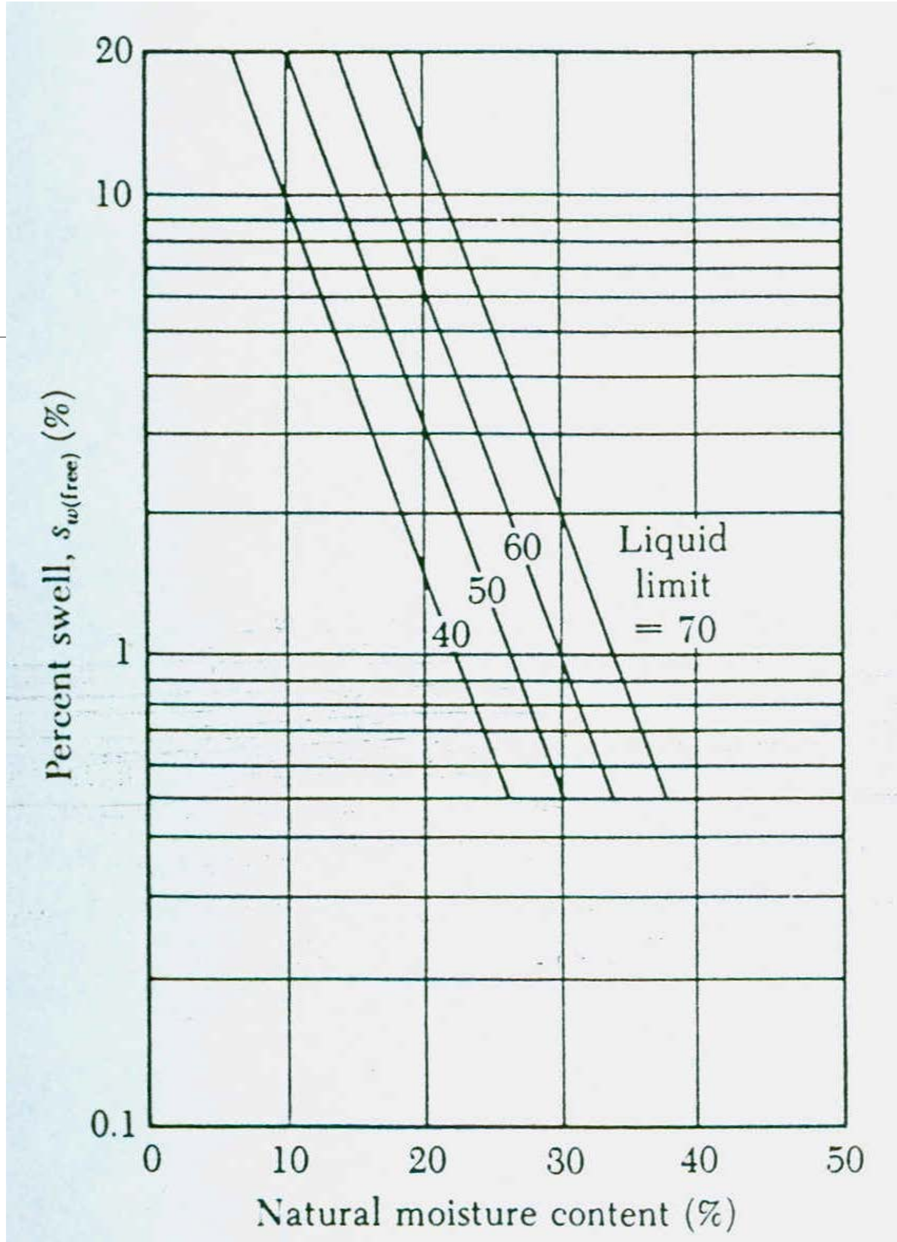
Expansive Soils

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

where ΔS_F = free surface swell

Z = depth of active zone

$s_{w(\text{free})}$ = free swell, as a percent



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

Expansive Soils

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

where V = soil volume after swelling

V_s = volume of soil solid = $\frac{W_s}{G_s \gamma_w}$

W_s = weight of oven-dried soil

G_s = specific gravity of soil solids

γ_w = unit weight of water

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

Sivapullaiah et al. (1987)

Expansive Soils

- A: Activity
- PI: Plasticity Index
- C: Clay Fraction
- w_I: Initial water Content
- SL: Shrinkage Index (Liquid Limit – Shrinkage Limit)
- LL: Liquid Limit
- γ_d : Dry Density
- B, m, k and M: Empirical Constant

Swelling Percent

Swelling Pressure

$Sp(\%) = Be^{A(PI)}$	Chen (1975)
$Sp(\%) = 7.518 + 0.323(C)$	Muntohar (2000)
$Sp(\%) = 60K(PI)^{2.44}$	Holtz et al (1956)
$Sp(\%) = 1.92A + 0.68w_i - 7.55$	J. Israr et al (2014)
$Sp(\%) = k(A^{2.44})(C^{3.44})$	Seed et al (1962)
$Sp(\%) = (k)(M)(PI)^{2.44}$	Seed et al (1962)
$Sp(\%) = m(SI)^{2.67}$	Ranganatham (1965)
$Log(P) = 2.132 + 0.0208(LL) + 0.000665(\gamma_d) - 0.0269(w_i)$	Komornic and David (1969)
$P = (3.5817 \times 10^{-2})(PI)^{1.12} C^2 / w_i^{\ 2} + 3.7912$	N. V. Nayak (1971)
$Log(P) = -4.812 + 0.01405PI + 2.394\gamma_d - 0.0163w_i$	Yusuf Erzin et al (2004)
$Log(P) = -5.197 + 0.01457PI + 2.408\gamma_d - 0.0163I_L$	Yusuf Erzin et al (2004)

Empirical correlations for predicting the swelling behavior of expansive soils by various researchers.

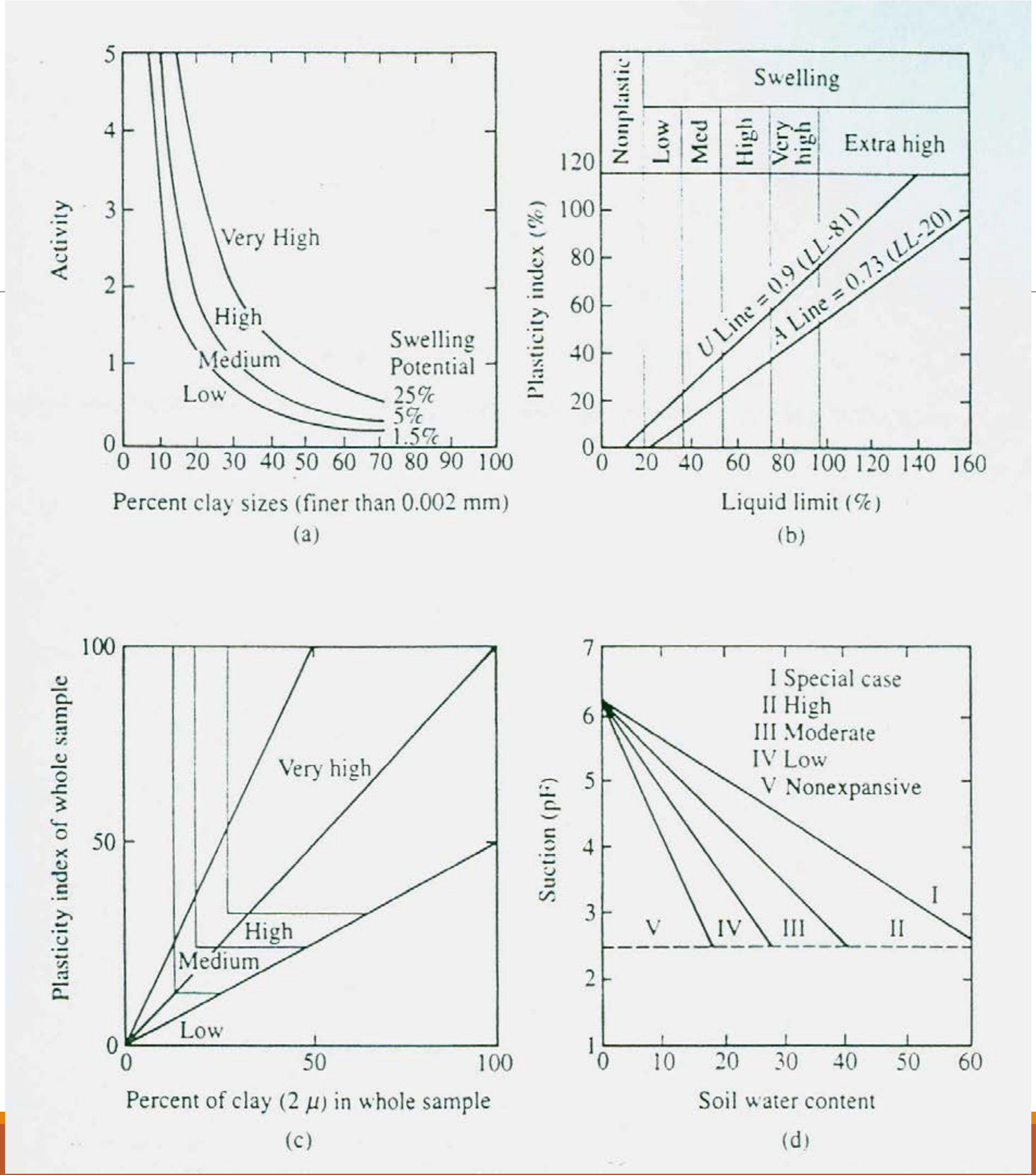
Expansive Soils

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

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

ΔH_i = thickness of layer i

Expansive Soils



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

Expansive Soils – Precautions

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