CHAPTER 5
APPLICATIONS WITH UNSTABLE FOUNDATION SOIL

5-1. Unstable Soils. Many types of soils change volume from causes different from elastic deformation, consolidation, and secondary compression. These volume changes cause excessive total and differential movements of overlying structures and embankments in addition to load induced settlement of the soil. Such unstable conditions include the heaving of expansive clays and collapse of silty sands, sandy silts, and clayey sands from alteration of the natural water content. Refer to Chapter 6 for coping with movements.

a. Effects of Excessive Movements. Excessive total and, especially, differential movements have caused damages to numerous structures that have not been adequately designed to accommodate the soil volume changes. Types of damage include impaired functional usefulness of the structure, external and interior cracked walls, and jammed and misaligned doors and windows. Important factors that lead to damages are the failure to recognize the presence of unstable soil and to make reasonable estimates of the magnitude of maximum heave or settlement/collapse. Adequate engineering solutions such as special foundation designs and soil stabilization techniques exist to accommodate the anticipated soil movement. A thorough field investigation is necessary to properly assess the potential movement of the soil. A qualitative estimate of potential vertical movement of proposed new construction may sometimes be made by examination of the performance of existing structures adjacent to the new construction.

b. Influence of Time on Movement. The time when heave or settlement/collapse occurs cannot be easily predicted because the location and time when water becomes available to the foundation soil cannot readily be foreseen. Heave or settlement can occur almost immediately in relatively pervious foundation soil, particularly in local areas subject to poor surface drainage and in soil adjacent to leaking water lines. More often, heave or settlement will occur over months or years depending on the availability of moisture. Soil movement may be insignificant for many years following construction permitting adequate performance until some change occurs in field conditions to disrupt the moisture regime. Predictions of when heave or settlement occurs is usually of little engineering significance. Important engineering problems include reliable determination of the magnitude of potential heave or settlement and development of ways to minimize this potential for movement and potential distress of the structure.

Section I. Heaving Soil

5-2. General. Expansive or swelling soils are found in many areas throughout the United States and the entire world. These soils change volume within the active zone for heave from changes in soil moisture. Refer to TM 5-818-7, Foundations in Expansive Soil, for details on mechanisms of heave, analysis and design of foundations in expansive soil.

a. Soils Susceptible to Heave. These soils consist of plastic clays and clay shales that often contain colloidal clay minerals such as the montmorillonites or smectite. They include marls, clayey siltstone and sandstone, and saprolites. Some soils, especially dry residual clayey soil, may heave on wetting under low applied pressure, but collapse at higher pressure. Other clayey soil may initially collapse on wetting, but heave over long periods of time as water slowly wets the less pervious clay particles. Desiccation can cause expansive soil to shrink.

b. Depth of Active Zone. The depth of the active zone Z_a illustrated in Figure 5-1 is defined as the least soil depth above which changes in water content, and soil heave may occur because of change in environmental conditions following construction. The water content distribution should not change with time below Z_a. Experience indicates Z_a may be approximated following guidelines in Table 5-1.

c. Equilibrium Pore Water Pressure Profile. The pore water pressure beneath the center of the foundation is anticipated to reach an equilibrium distribution; whereas, the pore water pressure profile beneath the perimeter will cycle between dry and wet extremes depending on the availability of water and the climate. Placement of a foundation on the soil may eliminate or reduce evaporation of moisture from the ground surface and eliminate transpiration of moisture from previously existing vegetation. Figure 5-1 illustrates three methods described below for estimating the equilibrium pore water pressure profile u_{wf} in units of tsf. If undisturbed soil specimens are taken from the field near the end of the dry season, then the maximum potential heave may be estimated from results of swell tests performed on these specimens.

(1) Saturated profile (Method 1, Figure 5-1). The equilibrium pore water pressure in the saturated profile within depth Z_a is

$$u_{wf} = 0 \tag{5-1a}$$

This profile is considered realistic for most practical cases including houses or buildings exposed to watering of perimeter vegetation and possible leaking of underground water and sewer lines. Water may also condense or collect in permeable soil beneath foundation slabs and penetrate into underlying expansive soil unless drained away or protected by a moisture barrier. This profile should be used if other information on the equilibrium pore water pressure profile is not available.

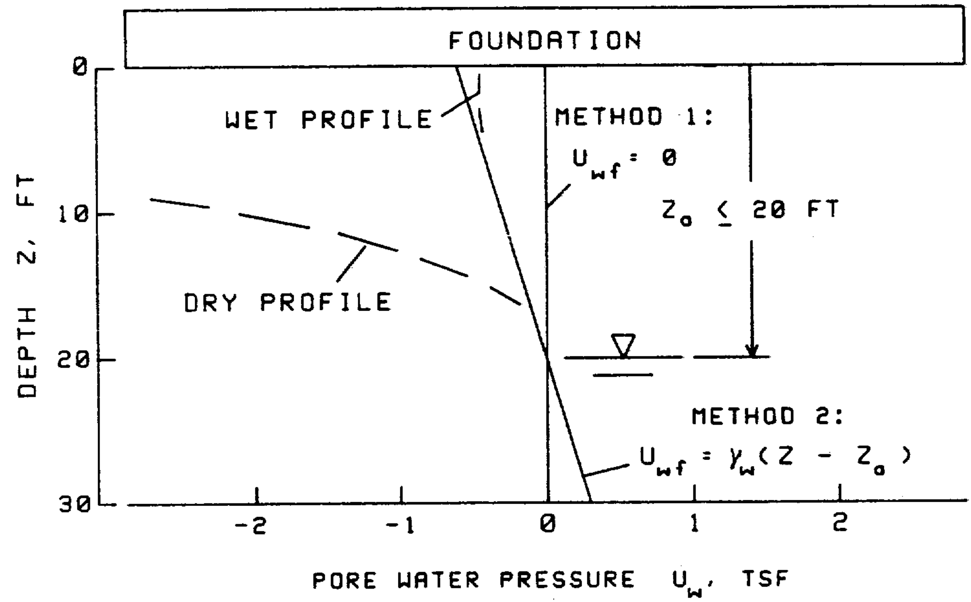
(2) Hydrostatic with shallow water table (Method 2, Figure 5-1). The equilibrium pore water pressure in this profile is zero at the groundwater level and decreases linearly with increasing distance above the groundwater level in proportion to the unit weight of water

$$u_{wf} = \gamma_w(z - Z_a) \tag{5-1b}$$

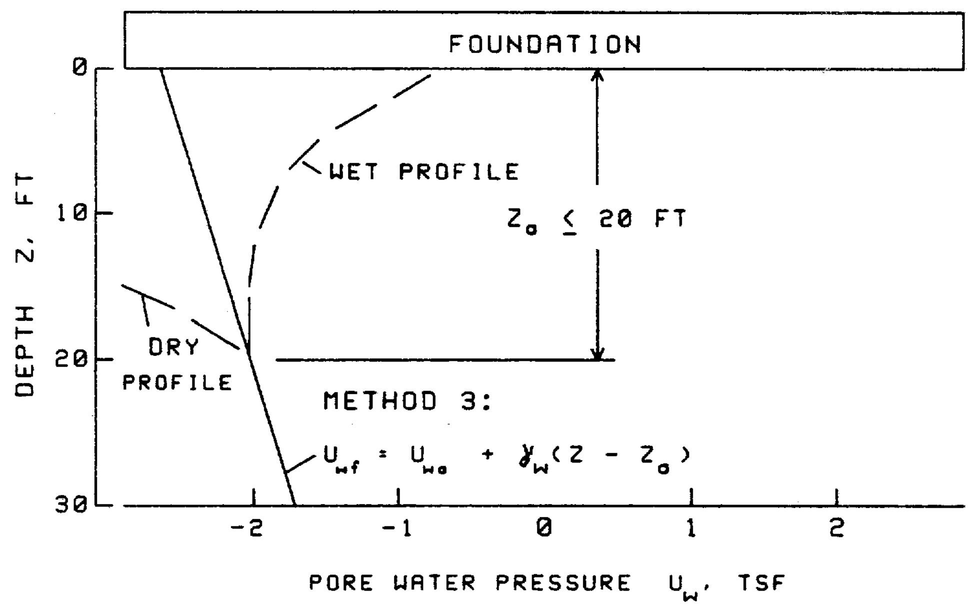
where

γ_w = unit weight of water, 0.031 ton/ft^3
z = depth below the foundation, ft

This profile is considered realistic beneath highways and pavements where surface water is drained from the pavement and where underground sources of water such as leaking pipes or drains do not exist. This assumption leads to smaller estimates of anticipated heave than Method 1.



a. SHALLOW GROUNDWATER LEVEL



b. DEEP GROUNDWATER LEVEL

Figure 5-1. Anticipated equilibrium pore water pressure profiles

Table 5-1

Guidelines For Estimating Depth of the Active Zone Z_a

Relative To	Guideline
Water table	Z_a will extend to depths of shallow groundwater levels $\leq$ 20 ft
Swell pressure	Z_a will be located within depths where $\sigma_{sj} - \sigma_{fj} \geq 0$, σ_{sj} = average swell pressure of stratum j and σ_{fj} = total average vertical overburden pressure after construction in stratum j
Fissures	Z_a will be within the depth of the natural fissure system caused by seasonal swell/shrinkage
Climate	Z_a, ft
	humid 10
	semi-arid 15
	arid 20

(3) Hydrostatic without shallow water table (Method 3, Figure 5-1). The pore water pressure of this profile is similar to Method 2, but includes a value of the negative pore water pressure u_{wa} at depth Z_a .

$$u_{wf} = u_{wa} + \gamma_w (z - Z_a) \tag{5-1c}$$

u_{wa} may be evaluated by methodology described in TM 5-818-7.

5-3. Identification. Soils susceptible to swelling can be most easily identified by simple classification tests such as Atterberg limits and natural water content. Two equations that have provided reasonable estimates of free swell are (item 30,64)

$$\log S_f = 0.0367LL + 0.0833W_n + 0.458 \tag{5-2a}$$

and (item 49)

$$S_f = 2.27 + 0.131LL - 0.27W_n \tag{5-2b}$$

where

S_f = free swell, percent
LL = liquid limit, percent
W_n = natural water content, percent

The percent swell under confinement can be estimated from the free swell by (item 20)

$$S = S_f \cdot (1 - 0.72\sqrt{\sigma_f}) \tag{5-3}$$

where

S = swell under confinement, percent
σ_f = vertical confining pressure, tsf

These identification procedures were developed by correlations of classification test results with results of 1-D swell tests performed in consolidometers on undisturbed and compacted soil specimens. Soils with liquid limit less than 35 percent and plasticity index less than 12 percent have relatively low potential for swell and may not require swell testing. Refer to TM 5-818-7 for further details on identification of expansive soils.

5-4. Potential Vertical Heave. Useful estimates of the anticipated heave based on results from consolidometer swell tests can often be made.

a. Selection of Suitable Test Method. Suitable standard test methods for evaluating the potential for 1-D heave or settlement of cohesive soils are fully described in EM 1110-2-1906 and ASTM D 4546. A brief review of three 1-D consolidometer tests useful for measuring potential swell or settlement using a standard consolidometer illustrated in Figure E-1, Appendix E, is provided below.

(1) Free swell. After a seating pressure (e.g., 0.01 tsf applied by the weight of the top porous stone and load plate) is applied to the specimen in a consolidometer, the specimen is inundated with water and allowed to swell vertically until primary swell is complete. The specimen is loaded following primary swell until its initial void ratio/height is obtained. The total pressure required to reduce the specimen height to the original height prior to inundation is defined as the swell pressure σ_s .

(2) Swell overburden. After a vertical pressure exceeding the seating pressure is applied to the specimen in a consolidometer, the specimen is inundated with water. The specimen may swell, swell then contract, contract, or contract then swell. The vertical pressure is often equivalent to the in situ overburden pressure and may include structural loads depending on the purpose of the test.

(3) Constant volume. After a seating pressure and additional vertical pressure, often equivalent to the in situ overburden pressure, is applied to the specimen in a consolidometer, the specimen is inundated with water. Additional vertical pressure is applied as needed or removed to maintain a constant height of the specimen. A consolidation test is subsequently performed as described in Appendix E. The total pressure required to maintain a constant height of the specimen is the measured swell pressure. This measured swell pressure is corrected to compensate for sample disturbance by using the results of the subsequent consolidation test. A suitable correction procedure is similar to that for estimating the maximum past pressure.

b. Calculation From Void Ratio. The anticipated heave is

$$S_{max} = \sum_{j=1}^{n} S_{maxj} = \sum_{j=1}^{n} \frac{e_{fj} - e_{oj}}{1 + e_{oj}} \bullet H_j \tag{5-4a}$$

where

S_{max} = maximum potential vertical heave, ft
n = number of strata within the depth of heaving soil
S_{maxj} = heave of soil in stratum j, ft
H_j = thickness of stratum j, ft
e_{fj} = final void ratio of stratum j
e_{oj} = initial void ratio of stratum j

The initial void ratio, which depends on a number of factors such as the maximum past pressure, type of soil, and environmental conditions, may be measured by standard consolidometer test procedures described in EM 1110-2-1906 or ASTM D 4546. The final void ratio depends on changes in soil confinement pressure and water content following construction of the structure; it may be anticipated from reasonable estimates of the equilibrium pore water pressure u_{wf}, depth of active zone Z_a, and edge effects by rewriting Equation 5-4a in terms of swell pressure shown in Equation 5-4b below.

c. Calculation from Swell Pressure. The anticipated heave in terms of swell pressure is

$$S_{max} = \sum_{j=1}^{n} \frac{C_{sj}}{1 + e_{oj}} \bullet \log_{10} \frac{\sigma_{sj}}{\sigma'_{fj}} \bullet H_j \tag{5-4b}$$

where

C_{sj} = swell index of stratum j
σ_{sj} = swell pressure of stratum j, tsf
σ'_{fj} = final or equilibrium average effective vertical pressure of stratum j, $\sigma_{fj} - u_{wfj}$, tsf
σ_{fj} = final average total vertical pressure of stratum j, tsf
u_{wfj} = average equilibrium pore water pressure in stratum j, tsf

The number of strata n required in the calculation is that observed within the depth of the active zone for heave.

(1) Swell index. The swell or rebound index of soil in each stratum may be determined from results of consolidometer tests as described in Section III, Chapter 3, and Figure 3-13. Preliminary estimates of the swell index may be made from Figure 3-14.

(a) The swell index C_s measured from a swell overburden test (Swell Test described in EM 1110-2-1906 or Method B described in ASTM D 4546 may be less than that measured from a constant volume test (Swell Pressure Test described in EM 1110-2-1906 or Method C described in ASTM D 4546). The larger values of C_s are often more appropriate for analysis of potential heave and design.

(b) A simplified first approximation of C_s developed from Corps of Engineer project sites through Central Texas is $C_s \approx 0.03 + 0.002(LL-30)$.

(2) Swell pressure. The swell pressure of soil in each stratum may be found from results of consolidometer swell tests on undisturbed specimens as described in EM 1110-2-1906 or ASTM D 4546. Preliminary estimates of swell pressure may be made from (item 32)

$$\log_{10}\sigma_s = \bar{2}.1423 + 0.0208LL + 0.01065\gamma_d - 0.0269W_n \tag{5-5a}$$

where

σ_s = swell pressure, tsf
γ_d = dry density, lbs/ft^3

An alternative equation (item 46) is

$$\sigma_s = 0.00258 \bullet PI^{1.12} \bullet \left[\frac{C}{W_n}\right]^2 + 0.273 \tag{5-5b}$$

where

PI = plasticity index, percent
C = clay content, percent less than 2 microns

(3) Final effective vertical pressure. The final total pressure σ_f may be estimated from the sum of the increase in soil stresses from the structural loads calculated by methods in Appendix C or Figure 1-2 and the initial overburden pressure σ_o . The final effective pressure σ_f' is σ_f less the assumed equilibrium pore water pressure profile u_{wf} , Figure 5-1.

5-5. Potential Differential Heave. Differential heave results from edge effects beneath a finite covered area, drainage patterns, lateral variations in thickness of the expansive foundation soil, and effects of occupancy. The shape, geometry, and loads of the structure also promote differential movement. Examples of the effect of occupancy include broken or leaking underground water lines and irrigation of vegetation adjacent to the structure. Other causes of differential heave include differences in distribution of loads and footing sizes.

a. Predictability of Variables. Reliable estimates of the anticipated differential heave and location of differential heave are not possible because of uncertainty in such factors as future availability of moisture, horizontal variations in soil parameters, areas of soil wetting, and effects of future occupancy.

b. Magnitude of Differential Heave. The difference in potential heave between locations beneath a foundation can vary from zero to the maximum potential vertical heave. Differential heave is often the anticipated total heave for structures on isolated spot footings or drilled shafts because soil beneath some footings or portions of slab foundations may experience no wetting and no movement. Refer to Chapter 2 for details on effect of differential movement on performance of the foundation.

(1) A reasonable estimate of the maximum differential movement or differential potential heave ΔS_{max} is the sum of the maximum calculated settlement ρ_{max} of soil beneath a nonwetted point of the foundation and the maximum potential heave S_{max} following wetting of soil beneath some adjacent point of the foundation separated by the distance ℓ . If all of the soil

heaves, then ΔS_{max} is the difference between S_{max} and S_{min} between adjacent points where S_{min} is the minimum heave.

(2) The location of S_{max} may be beneath the most lightly loaded portion of the foundation such as beneath the center of the slab.

(3) The location of ρ_{max} may be beneath columns and consist only of immediate elastic settlement ρ_i in soil where wetting does not occur or will be S_{min} if wetting does occur in expansive soil.

(4) The deflection ratio is $\Delta S_{max}/L$ where L may be the distance between stiffening beams.

5-6. Application. A stiffened ribbed mat is to be constructed on an expansive soil. The soil parameters illustrated in Table 5-2 were determined on specimens of an undisturbed soil sample taken 10 ft beneath the mat. Additional tests at other depths will improve reliability of these calculations. The active zone for heave is estimated to extend 20 ft below ground surface or 20 ft below the base of the mat and 17 ft below the base of the columns. The maximum anticipated heave S_{max} and differential heave ΔS_{max} are to be estimated beneath portions of the mat. Stiffening beams are 3 ft deep with 20-ft spacing in both directions, Figure 5-2. Column loads of 25 tons interior and 12.5 tons perimeter lead to an applied pressure on the column footings q = 1.0 tsf. Minimum pressure q_{min} beneath the 5-inch-thick-flat slab is approximately 0.05 tsf. The heave calculations assume a zero stiffness mat. Computer program VDISPL in Appendix F is useful for calculating potential heave beneath footings and mat foundations in multilayered expansive soil. VDISPL also considers heave in an excavation from changes in pore water pressure.

a. Calculation of Potential Heave.

(1) Maximum potential heave S_{max} . The maximum heave is anticipated beneath unloaded portions of the mat. The potential heave is estimated assuming the equilibrium pore water pressure $u_{wf} = 0$ or the soil is saturated; therefore, the final effective pressure $\sigma_f' = \sigma_f$ or the final total pressure.

(a) Table 5-3a illustrates the estimation of anticipated heave S_{max} beneath lightly loaded portions of the mat using Equation 5-4b, increment thickness ΔH = 2 ft , and results of a single consolidometer swell test.

(b) Table 5-3a and Figure 5-3a show that S_{max} = 0.3 ft or 3.6 inches and that heave is not expected below 16 ft of depth where the swell pressure approximately equals the total vertical pressure σ_f .

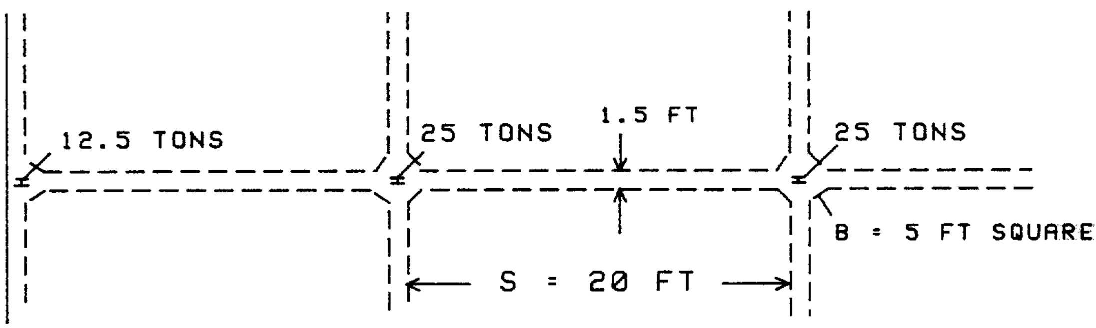
(c) Most heave occurs at depths less than 5 ft below the flat portion of the mat. Replacing the top 4 ft of expansive soil with nonexpansive backfill will reduce S_{max} to 0.115 ft or 1.4 inches, Table 5-3a and Figure 5-3a.

(2) Minimum potential heave S_{min} . The minimum potential heave on wetting of the soil to a saturated profile (Method 1, Figure 5-1) is expected beneath the most heavily loaded portions of the mat or beneath the columns. Table 5-3b and Figure 5-3b show that the minimum heave S_{min} calculated after Equation 5-4b substituting S_{min} for S_{max} is 0.092 ft beneath the column or

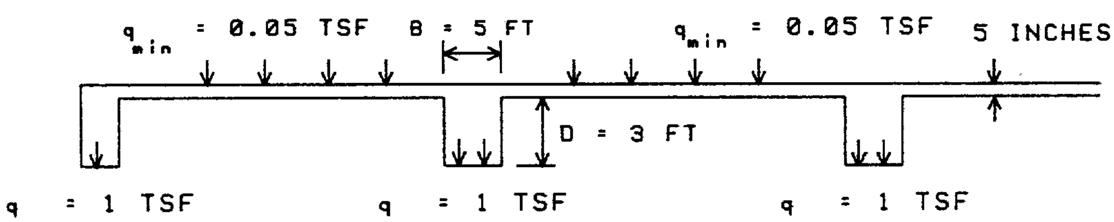
Table 5-2

Soil Parameters For Example Estimation of Anticipated Heave

Parameter	Value
Elastic modulus E_s , tsf	200
Swell Pressure σ_s , tsf	1.0
Compression index C_c	0.25
Swell index C_s	0.10
Initial void ratio e_o	0.800
Unit wet soil weight γ, ton/ft^3	0.06
Active zone for heave Z_a , ft	20



a. PLAN



b. ELEVATION

Figure 5-2. Plan and elevation of stiffened mat in expansive soil

Table 5-3

Heave Calculations for Example Application

a. Beneath Slab

Depth z, ft	Overburden Pressure $\sigma_o = \gamma z$, tsf	Total Pressure $\sigma_f = \sigma'_f$, tsf	$\frac{S_{maxj}}{\Delta H}$	S_{maxj}, ft	S_{max}, ft
0	0.00	0.05	0.072		0.303
				0.115	
2	0.12	0.17	0.043		0.188
				0.073	
4	0.24	0.29	0.030		0.115
				0.052	
6	0.36	0.41	0.022		0.063
				0.037	
8	0.48	0.53	0.015		0.026
				0.025	
10	0.60	0.65	0.010		0.001
				0.016	
12	0.72	0.77	0.006		-0.015
				0.009	
14	0.84	0.89	0.003		-0.024
				0.003	
16	0.96	1.01	0.000		-0.027
				-0.007	
18	1.08	1.13	-0.007		-0.020
				-0.020	
20	1.20	1.25	-0.013		0.000

b. Beneath Columns

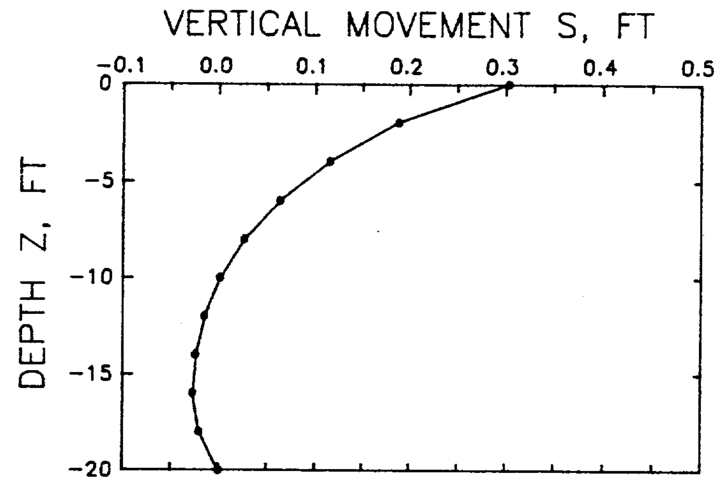
Depth z, ft	$\frac{z}{B}$	Overburden Pressure $\sigma_o = \gamma z$, tsf	Column* Pressure $\Delta\sigma_z$, tsf	Total Pressure $\sigma_f = \sigma'_f$, tsf	$\frac{S_{minj}}{\Delta H}$	S_{minj}, ft	S_{min}, ft
0	0.0	0.00	1.00	1.00	0.000		0.092
						-0.003	
1	0.2	0.06	0.96	1.02	-0.006		0.095
						0.000	
3	0.6	0.18	0.61	0.79	0.006		0.095
						0.018	
5	1.0	0.30	0.30	0.60	0.012		0.077
						0.024	
7	1.4	0.42	0.20	0.62	0.012		0.053
						0.021	
9	1.8	0.54	0.16	0.70	0.009		0.032
						0.016	
11	2.2	0.66	0.09	0.75	0.007		0.016
						0.011	
13	2.6	0.78	0.07	0.85	0.004		0.005
						0.005	
15	3.0	0.90	0.05	0.95	0.001		0.000
						0.000	
17	3.4	1.02	0.04	1.06	-0.001		0.000

* Increase in pressure beneath columns calculated from Figure 1-2, Table C-1a (point under corner rectangular area) where $\Delta\sigma_z = 4q$ or Figure C-2 where $\Delta\sigma_z = 4q$.

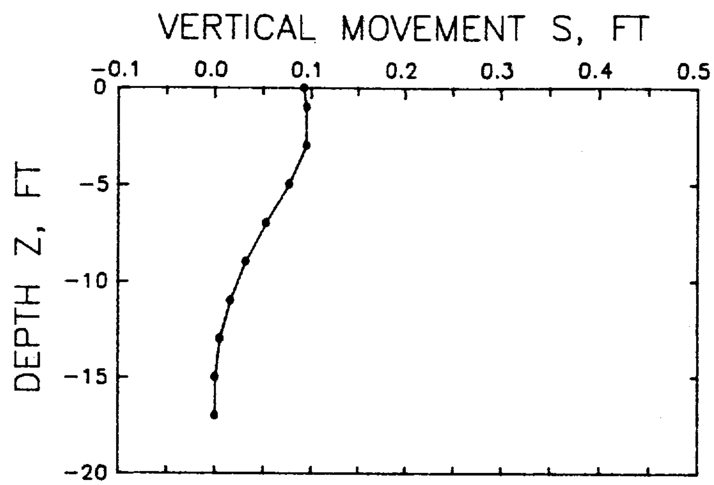
S_{min} is 0.092 ft or 1.1 inches beneath the column. Heave is not expected below 13 ft beneath the columns.

b. Maximum Differential Heave ΔS_{max}.

(1) ΔS_{max} is the sum of S_{max} and the immediate settlement ρ_i if soil wetting is nonuniform. The maximum immediate settlement ρ_i is anticipated to occur as elastic settlement beneath the loaded columns if soil wetting does not occur in this area. A common cause of nonuniform wetting is



a. **BENEATH SLAB**



b. **BENEATH COLUMN**

Figure 5-3. Calculated heave profile beneath mat foundation

leaking underground water lines. From the improved Janbu approximation, Equation 3-17 and Figure 3-8, with reference to Figure 5-2

$$\mu_o = 0.92 \textit{ for } D/B = 1.0$$

$$\mu_1 = 0.7 \textit{ for } L/B = 1.0 \textit{ and } H/B > 10$$

$$\rho_i = \mu_o \bullet \mu_1 \bullet \left[\frac{q \bullet B}{E_s}\right] = 0.92 \bullet 0.7 \bullet \frac{1.0 \bullet 5.0}{200}$$

The maximum differential heave $\Delta S_{max} = S_{max} + \rho_i = 3.6 + 0.2 = 3.8$ inches or 0.317 ft. The deflection ratio Δ/L is $\Delta S_{max}/L = 0.317/20$ or 1/64 where L is 20 ft, the stiffening beam spacing. This deflection ratio cannot be tolerated, Chapter 2. If the top 6 ft of expansive soil is replaced with nonexpansive backfill $\Delta S_{max} = 0.063 + 0.016 = 0.079$ ft or 0.95 inch. Ribbed mat foundations and superstructures may be designed to accommodate differential heave of 1 inch after methods in TM 5-818-7 or item 28.

(2) ΔS_{max} is the difference between S_{max} and S_{min} if soil wetting occurs beneath the columns or 3.6 - 1.4 = 2.2 inches. Replacement of the top 4 ft of soil beneath the ribbed mat will reduce this differential heave to about 1.4 - 1.1 or about 0.3 inch ignoring the difference in settlement beneath the fill and original expansive soil within 1 ft beneath the column.

Section II. Collapsible Soil

5-7. General. Many collapsible soils are mudflows or windblown silt deposits of loess often found in arid or semiarid climates such as deserts, but dry climates are not necessary for collapsible soil. Loess deposits cover parts of the western, midwestern, and southern United States, Europe, South America, Asia including large areas of Russia and China, and Southern Africa. A collapsible soil at natural water content may support a given foundation load with negligible settlement, but when water is added to this soil the volume can decrease significantly and cause substantial settlement of the foundation, even at relatively low applied stress or at the overburden pressure. The amount of settlement depends on the initial void ratio, stress history of the soil, thickness of the collapsible soil layer, and magnitude of the applied foundation pressure. Collapsible soils exposed to perimeter watering of vegetation around structures or leaking utility lines are most likely to settle. Collapse may be initiated beneath the ground surface and propagate toward the surface leading to sudden and nonuniform settlement of overlying facilities.

a. Structure. Soils subject to collapse have a honeycombed structure of bulky shaped particles or grains held in place by a bonding material or force illustrated in Figure 5-4. Common bonding agents include soluble compounds such as calcareous or ferrous cementation that can be weakened or partly dissolved by water, especially acidic water. Removal of the supporting material or force occurs when water is added enabling the soil grains to slide or shear and move into voids.

b. Collapse Trigger. Table 5-4 illustrates four types of wetting that can trigger the collapse of soil. Dynamic loading may also cause a shear failure in the bonding material and induce collapse. This mechanism is particularly important for roads, airfields, railways, foundations supporting vibrating machinery, and other foundations subject to dynamic forces.

5-8. Identification. Typical collapsible soils are lightly colored, low in plasticity with liquid limits below 45, plasticity indices below 25, and relatively low dry densities between 65 and 105 lbs/ft^3 (60 to 40 percent porosity). Collapse rarely occurs in soil with a porosity less than 40 percent. Most past criteria for determining the susceptibility of collapse are based on relationships between the void ratio, water content, and dry density, Table 5-5. The methods in Table 5-5 apply to fine-grained soil.

(1) The Gibbs and Bara method (item 18) assumes collapse of soil with sufficient void space to hold the liquid limit water.

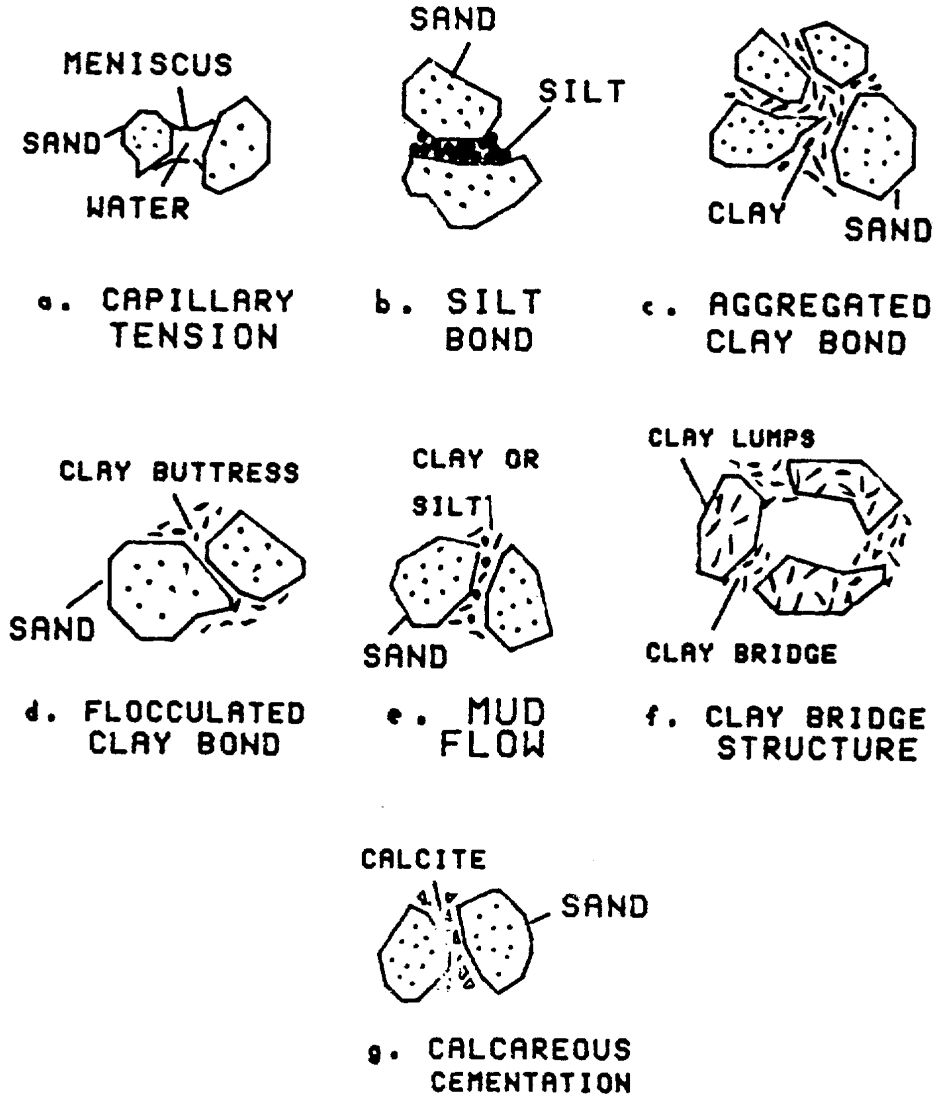


Figure 5-4. Mechanisms for collapse of loose, bulky grains

(2) Fine-grained soils that are not susceptible to collapse by the criteria in Table 5-5 may have potential for expansion described in Section I of this chapter.

5-9. Potential Collapse. When water becomes available to collapsible soil, settlement in addition to elastic settlement will occur without any additional applied pressure. This settlement will occur quickly in a free draining or pervious soil, but more slowly in a poor draining or less pervious soil. When construction occurs on soil where surface water filters through the collapsible soil over time, some collapse will occur in situ and reduce collapse that will occur on wetting following construction. Procedures for estimating the potential for collapse are uncertain because no single criterion can be applied to all collapsible soil. The amount of settlement depends on the extent of the wetting front and availability of water, which rarely can be predicted prior to collapse. Laboratory classification and consolidation tests can fail to indicate soil that eventually does collapse in the field. The following

Table 5-4

Wetting That Can Trigger Soil Collapse

Type of Wetting	Description
Local, shallow	Wetting of a random nature caused by water sources from pipelines or uncontrolled drainage of surface water; no rise in groundwater level; settlement occurs in upper soil layer within wetted area
Intense local	Intense deep, local wetting caused by discharge of deep industrial effluent, leaking underground utility lines, or irrigation. Flow rates sufficient to cause a continuous rise in groundwater level may saturate the entire zone of collapsible soil within a short time (i.e., < 1 year) and cause uneven and damaging settlement under existing structural loads or only the soil weight
Slow, uniform rise in groundwater	Slow, relatively uniform rise of groundwater from sources outside of the collapsible soil area will cause uniform and gradual settlement
Slow increase in water content	Gradual increase in water content of thick collapsible soil layer from steam condensation or reduction in evaporation from the ground surface following placement of concrete or asphalt will cause incomplete settlement

procedures to estimate collapse attempt to follow the stress path to which the soil will be subjected in the field. Immediate settlement prior to collapse may be estimated by methods in Sections I and II, Chapter 3.

a. Wetting at Constant Load. An acceptable test procedure is described in detail as Method B of ASTM D 4546 or the Swell Test Procedure in Appendix VIIIA of EM 1110-2-1906. A specimen is loaded at natural water content in a consolidometer to the anticipated stress that will be imposed by the structure in the field. Distilled water (or natural site water if available) is added to the consolidometer and the decrease in specimen height following collapse is noted. The settlement of collapsible soil may be estimated by

$$\rho_{col} = \frac{e_o - e_c}{1 + e_o} \bullet H \qquad (5\text{-}6)$$

where

ρ_{col} = settlement of collapsible soil stratum, ft
e_o = void ratio at natural water content under anticipated vertical applied pressure σ_f
e_c = void ratio following wetting under σ_f
H = thickness of collapsing soil stratum, ft

Table 5-5

Relationships for Estimating Susceptibility
of Soil to Collapse

Source	Soil Susceptible to Collapse
Northey 1969 (item 48)	Denisov introduced a coefficient of subsidence $k_d = e_{LL}/e_o$; the soil is collapsible if $0.5 < k_d < 0.75$ where e_{LL} = void ratio at liquid limit LL = limit limit, percent e_o = natural void ratio
After Gibbs and Bara 1962 (item 18)	$\gamma_d < 162.3/(1 + 0.026LL)$ where γ_d = natural dry density, lbs/ft^3 or $e_o > 2.6LL/100$
Feda 1966 (item 16)	$\dfrac{\dfrac{e_o}{100G_s} - PL}{PI} > 0.85$ PL = plastic limit, percent PI = plasticity index, percent G_s = specific gravity of soil
Jennings and Knight 1975 (item 26)	Measure of collapse potential CP of a specimen tested in a one-dimensional consolidometer in percent $CP = \dfrac{e_o - e_c}{1 + e_o} \bullet 100$ e_o = void ratio at σ = 2 tsf at natural water content prior to wetting e_c = void ratio after soaking at σ - 2 tsf

CP Percent	Severity of of Collapse
0 - 1	negligible
1 - 5	moderate trouble
5 - 10	trouble
10 - 20	severe trouble
> 20	very severe trouble

The total settlement of the soil will be the sum of the settlement of each stratum.

b. Modified Oedometer Test (item 22). This test is a modification of the Jennings and Knight (item 26) double oedometer procedure that eliminates testing of two similar specimens, one at natural water content and the other inundated with distilled (or natural) water for 24 hr.

(1) Procedure. An undisturbed specimen is prepared and placed in a 1-D consolidometer at the natural water content. The initial specimen height h

is recorded. A seating pressure of 0.05 tsf is placed on the specimen and the dial gauge is zeroed (compression at stress levels less than 0.05 tsf is ignored). Within 5 minutes, the vertical stress is increased in increments of 0.05, 0.1, 0.2, 0.4 tsf, etc. until the vertical stress is equal to or slightly greater than that expected in the field following construction. For each increment, dial readings are taken every 1/2 hr until less than 0.1 percent compression occurs in 1 hr. The specimen is subsequently inundated with distilled (or natural) water and the collapse observed on the dial gauge is recorded. Dial readings are monitored every 1/2 hr at this stress level until less than 0.1 percent compression occurs in 1 hr. Additional stress is placed on the specimen in increments as previously described until the slope of the curve is established. The dial readings d are divided by the initial specimen thickness h_o and multiplied by 100 to obtain percent strain. The percent strain may be plotted as a function of the applied pressure as shown in Figure 5-5 and a dotted line projected from point C to point A to approximate the collapse strain for stress levels less than those tested.

(2) Calculation of collapse. The soil profile should be divided into different layers with each layer corresponding to a representative specimen such as illustrated in Figure 5-6. The initial and final stress distribution should be calculated for each layer and entered in the compression curve such as Figure 5-5 and the vertical strain recorded at the natural water content and the inundated water content. The settlement is the difference in strain between the natural water content and wetted specimen at the same stress level

$$\rho_{col} = \left[(d/h_o)_f - (d/h_o)_o\right] \bullet H/100 \qquad (5\text{-}7)$$

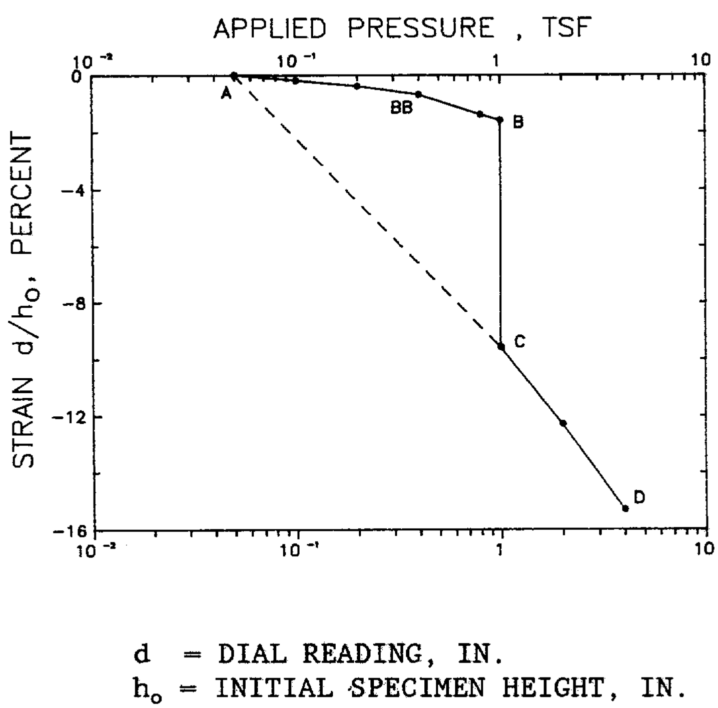


Figure 5-5. Example Compression curve of the Modified Oedometer Test. (d/h_o is multiplied by 100 to obtain percent)

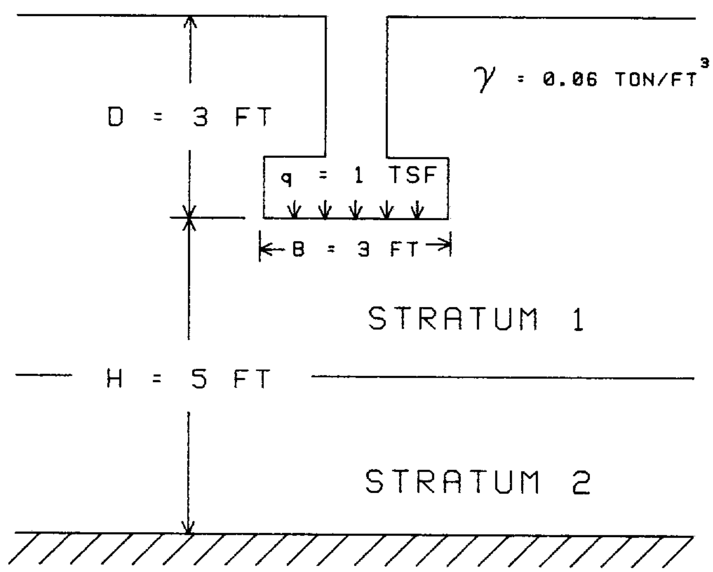


Figure 5-6. Illustration of footing for calculating settlement on collapsible soil

where

ρ_{col} = collapse settlement, ft
$(d/h_o)_f \cdot 100$ = strain after wetting at the field stress level, percent
$(d/h_o)_o \cdot 100$ = strain at natural water content at the field stress level, percent
d = dial reading, in.
h_o = initial specimen height, in.
H = thickness of collapsible stratum, ft

Total settlement is the sum of the collapse settlement of each stratum.

5-10. Application. A 3-ft square footing illustrated in Figure 5-6 is to be placed 3-ft deep on a loess soil with a thickness of 5 ft beneath the footing. The results of a modified oedometer test performed on specimens of this soil are provided in Figure 5-5. The footing pressure q = 1 tsf. Refer to Appendix F for calculation of potential collapse of a footing using program VDISPL.

a. Calculation. Table 5-6 illustrates computation of the vertical stress distribution and collapse settlement at the center and corner of this footing. The stress levels and vertical strains of the soil in Figure 5-5 are shown in Table 5-6b assuming layers 1 and 2, Figure 5-6, consist of the same soil. The average settlement of (4.5 + 4.0)/2 = 4.3 inches should provide a reasonable estimate of the settlement of this footing.

b. Testing Errors. The amount of collapse depends substantially on the extent of the wetting front and initial negative pore water or suction pressure in the soil, which may not be duplicated because soil disturbance and lateral pressures may not be simulated. Collapse may also be stress path dependent and may involve a mechanism other than addition of water such as exposure to dynamic forces.

Table 5-6

Example Calculation of Settlement of a
Collapsible Soil Beneath a Square Footing (Figure 5-6)

a. Stress Distribution

Depth Below Footing z, ft	$\frac{B}{2z}$	Overburden Pressure σ_o, tsf	Influence* Factor I_σ Cen	Influence* Factor I_σ Cor	Footing** Stress q_z, tsf Cen	Footing** Stress q_z, tsf Cor	Total Stress σ_{fz}, tsf Cen	Total Stress σ_{fz}, tsf Cor
0.0	∞	0.18	0.250	0.250	1.000	0.250	1.180	0.430
2.5	0.6	0.33	0.106	0.195	0.424	0.195	0.754	0.525
5.0	0.3	0.48	0.038	0.106	0.152	0.106	0.632	0.586

* From Figure C-2 where $m = n = \frac{B}{2z}$ for the center and $m = n = \frac{B}{z}$ for the corner

** Center: $q_z = 4q \cdot I$; Corner: $q_z = q \cdot I$; $q = 1$ tsf

b. Settlement

Depth Below Footing z, ft	Layer	Average Final Stress σ_{fz}, tsf Cen	Average Final Stress σ_{fz}, tsf Cor	Average Initial Strain, Percent Cen	Average Initial Strain, Percent Cor	Average Final Strain, Percent Cen	Average Final Strain, Percent Cor
1.25	1	0.967	0.478	1.55	0.85	9.45	7.30
3.75	2	0.693	0.555	1.25	1.05	8.35	7.75

Settlement from Equation 5-7:

Center: $\rho_{col} = [(9.45 - 1.55) + (8.35 - 1.250] \bullet 2.5 = 0.375\ ft\ or\ 4.5\ inches$

Corner: $\rho_{col} = [(7.30 - 0.85) + (7.75 - 1.05)] \bullet 2.5 = 0.329\ ft\ or\ 4.0\ inches$