# ENCE 4610 Foundation Design and Analysis

Lecture 20
Expansive and Collapsible Soils

TABLE 12
Identification and Characteristics of Special Materials

| Material | Geographic/Geomorphic Features | Engineering Conditions |
|---|---|---|
| "Quick Clay" | • Marine or brackish water clay composed of glacial rock flour that is elevated above sea level.<br>• Generally confined to far north areas; Eastern Canada, Alaska, Scandinavia. | • Severe loss of strength when disturbed by construction activities or seismic ground shaking.<br>• Replacement of formation water containing dissolved salt with fresh water results in strength loss.<br>• Produces landslide prone areas (Anchorage, Alaska). |
| Hydraulic Fills | • Coastal facilities, levees, dikes, tailings dams | • High void ratio<br>• Uniform gradation but variable grain sizes within same fill<br>• High liquefaction potential<br>• Lateral spreading<br>• Easily eroded |
| Collapsing Soil | • Desert arid and semi-arid environment<br>• Alluvial valleys, playas, loess | • Loss of strength when wetted<br>• Differential settlement<br>• Low density<br>• Moisture sensitive<br>• Gypsum/Anhydrite often present |

TABLE 12 (continued)
Identification and Characteristics of Special Materials

| Material | Geographic/Geomorphic Features | Engineering Conditions |
|---|---|---|
| Submarine Soils | • Continental shelf deposits at water depths up to several hundred feet.<br>• Submarine canyons, turbidity flows, deltaic deposits, abyssal plain | • Distribution and physical properties of sand, silt, and clay may change with time and local geologic conditions.<br>• Shelf deposits have few unique characteristics requiring modification of soil mechanics principals.<br>• Local areas, such as the Gulf of Mexico have weak, underconsolidated deposits.<br>• Deep sea calcareous deposits have water contents up to 100% and shear strengths up to about 220 psf.<br>• Deep sea silty clays have average water contents of 100-200% and shear strengths of 35-75 psf.<br>• Deep sea deposits are normally consolidated but near shelf deposits may be underconsolidated. |
| Lateritic Soils | • Tropical rainforest and savanna<br>• Deep residual soil profile<br>• Shield and sedimentary cover outside shield in South and Central America, Central and West Africa, southeast Asia, and other parts of the world. | • Loss of soil strength with time<br>• High void ratio/permeability<br>• Aggregate deterioration<br>• Variable moisture content<br>• Shrinkage cracks |

TABLE 12 (continued)
Identification and Characteristics of Special Materials

| Material | Geographic/Geomorphic Features | Engineering Conditions |
|---|---|---|
| Lateritic Soils (cont'd) | | • Easily compacts<br>• Shear characteristics somewhere between sand and silt<br>• Landslide prone<br>• Depth of wetting affects slope stability<br>• Varied foundation conditions |
| Limestone and Coral | • Humid tropics and subtropics, island environment.<br>• Karst topography accelerated in humid climates.<br>• Limestone that are cavernous or prone to cavity formations are widely distributed throughout the world in countries of arid and humid climates. In the U. S., cavernous limestone is found in Kentucky, Pennsylvania, California, Indiana, Michigan, New Mexico, Texas, and Virginia. | • Solution cavities<br>• Extreme variations in porosity<br>• Void ratios in coral up to 2<br>• Chimney-like sinkholes and collapse structures<br>• Slump failures, ravelling<br>• Rock settlement and consolidation<br>• Piles or bridging often required |

# Overview of Collapsible Soils

### 5.7.2 Collapse Potential of Soils

There are several types of soils that can experience collapse under moisture ingress. Examples of such soils are wind-blown deposits such as loess, or alluvial soils deposited in arid or semi-arid environments where evaporation of soil moisture takes place at such a rapid rate that the deposits do not have time to consolidate under their self weight or where the deposits are cemented by precipitated salts. Such soils are predominantly composed of silts and some clay. Typically, the structure of such soils is flocculated and the soil particles are held together by "clay bridges" or some other cementing agent such as calcium carbonate. In both cases disturbed samples obtained from these deposits are generally classified as silt. When dry or at low moisture content the in-situ material gives the appearance of a stable deposit. At elevated moisture contents these soils generally undergo sudden changes in volume and collapse. **Full saturation is not required to realize collapse of such soils; often collapse of the soil structure occurs at moisture contents corresponding to pre-collapse degrees of saturation between 50 to 70%.** Such soils, unlike other non-cohesive soils, will stand on almost a vertical slope until inundated. Collapse-susceptible soils typically have a low relative density, a low unit weight and a high void ratio. Figure 5-28 is a useful tool for assessing whether a soil is collapsible or not based on LL and dry unit weight.

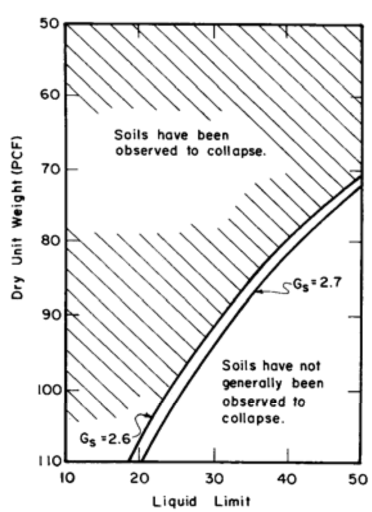



**Figure 5-28. Chart for evaluation of collapsible soils (after Holtz and Hilf, 1961).**

Structures founded on such soils may be seriously damaged if the soils are inundated and collapse. Therefore, if a soil is suspected to be collapse susceptible, then it is of primary importance to estimate the magnitude of potential collapse that may occur if the soil becomes wetted. To do this, a one-dimensional collapse potential test can be performed in an oedometer on undisturbed or recompacted samples according to ASTM D 5333. For this test, a sample is placed in an oedometer at it natural or compacted moisture content and the vertical pressure on the sample is increased in increments to the anticipated final loading in the field. Readings of vertical deformation are taken during the loading sequence. At the anticipated final load level, water is introduced to the sample and the resulting deformation due to collapse is recorded. The percent collapse (%C) is defined as:

$$\%C = \frac{100\,\Delta H_c}{H_o} \qquad \text{5-15}$$

where $\Delta H_c$ is the change in height upon wetting and $H_o$ is the initial height of the specimen. Conceptually, C is a strain. Therefore, for a soil layer with a given thickness, H, the settlement due to collapse, $s_{collapse}$, if the entire thickness is inundated may be calculated as:

$$s_{collapse} = H\left(\frac{\%C}{100}\right) \qquad \text{5-16}$$

The collapse potential (CP) is calculated as the percent collapse (%C) of a soil specimen subjected to a total load of 4 ksf (200 kPa) as measured by using procedures specified in ASTM D 5333. The CP is an index value used to compare the susceptibility of various soils to collapse. Table 5-12 provides a relative indication of the degree of severity for various values of CP.

**Table 5-12**
**Qualitative assessment of collapse potential (after ASTM D 5333)**

| Collapse Potential (CP) | Severity of Problem |
|---|---|
| 0 | None |
| 0.1 to 2% | Slight |
| 2.1 to 6% | Moderate |
| 6.1 to 10% | Moderately Severe |
| >10% | Severe |

# Dealing With Collapsible Soils

- Solutions for Pavements
  - Ponding water over the region of collapsible soils.
  - Infiltration wells.
  - Compaction - conventional with heavy vibratory roller for shallow depths (within 0.3 or 0.6 m (1 or 2 ft))
  - Compaction - dynamic or vibratory for deeper deposits of more than half a meter (a few feet) (could be combined with inundation)
  - Excavated and replaced.
- Other Foundation Solutions
  - Shallow Foundations—furnish a system of grade beams to distribute the load and mitigate the effects of uneven collapse of the underlying soils
  - Deep foundations—avoid the effects of collapsible soils altogether by transferring the structure load to a more stable stratum

# Expansive Soils

2. EXPANSIVE SOILS.

a. Characteristics. Expansive soils are distinguished by their potential for great volume increase upon access to moisture. Soils exhibiting such behavior are mostly montmorillonite clays and clay shales.

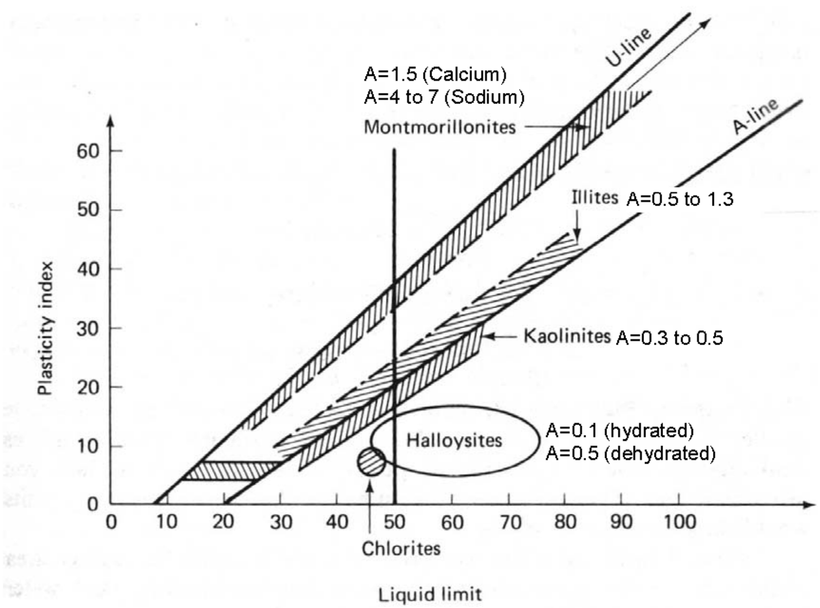
b. Identification and Classification. Figure 4 (Reference 17, Shallow Foundations, by the Canadian Geotechnical Society) shows a method based on Atterberg limits and grain size for classifying expansive soils. Activity of clay is defined as the ratio of plasticity index and the percent by weight finer than two microns ($2\mu$). The swell test in a one dimensional consolidation test (see Chapter 3) or the Double Consolidometer Test (Reference 18, The Additional Settlement of Foundations Due to Collapse of Structures of

# Activity Index

Another useful index proposed by Skempton (1953) based on the proportion of clay and PI is known as the "Activity Index." The activity index of a clay soil is denoted by A and is generally defined as follows:

$$A = \frac{PI}{CF} \qquad 5\text{-}3$$

where CF is the clay fraction is usually taken as the percentage by weight of the soil with a particle size less than 0.002 mm. Clays with $0.75 < A < 1.25$ are classified as "normal" clays while those with $A < 0.75$ are "inactive" and $A > 1.25$ are "active." Values of activity index, A, can be correlated to the type of clay mineral that, in turn, provides important information relative to the expected behavior of a clay soil. A clay soil that consists predominantly of the clay mineral montmorillonite behaves very differently from a clay soil composed predominantly of kaolinite. Figure 5-5 also shows the activities of various clay minerals and their location on the Casagrande's plasticity chart. The symbol for the activity index (A) in Figure 5-5 should not be confused with the "A-line" also shown in the figure.




| Activity Index, A, of other minerals | | | |
|---|---|---|---|
| Attapulgite | 0.5-1.2 | Mica (muscovite) | 0.2 |
| Allophane | 0.5-1.2 | Quartz | 0 |
| Calcite | 0.2 | | |

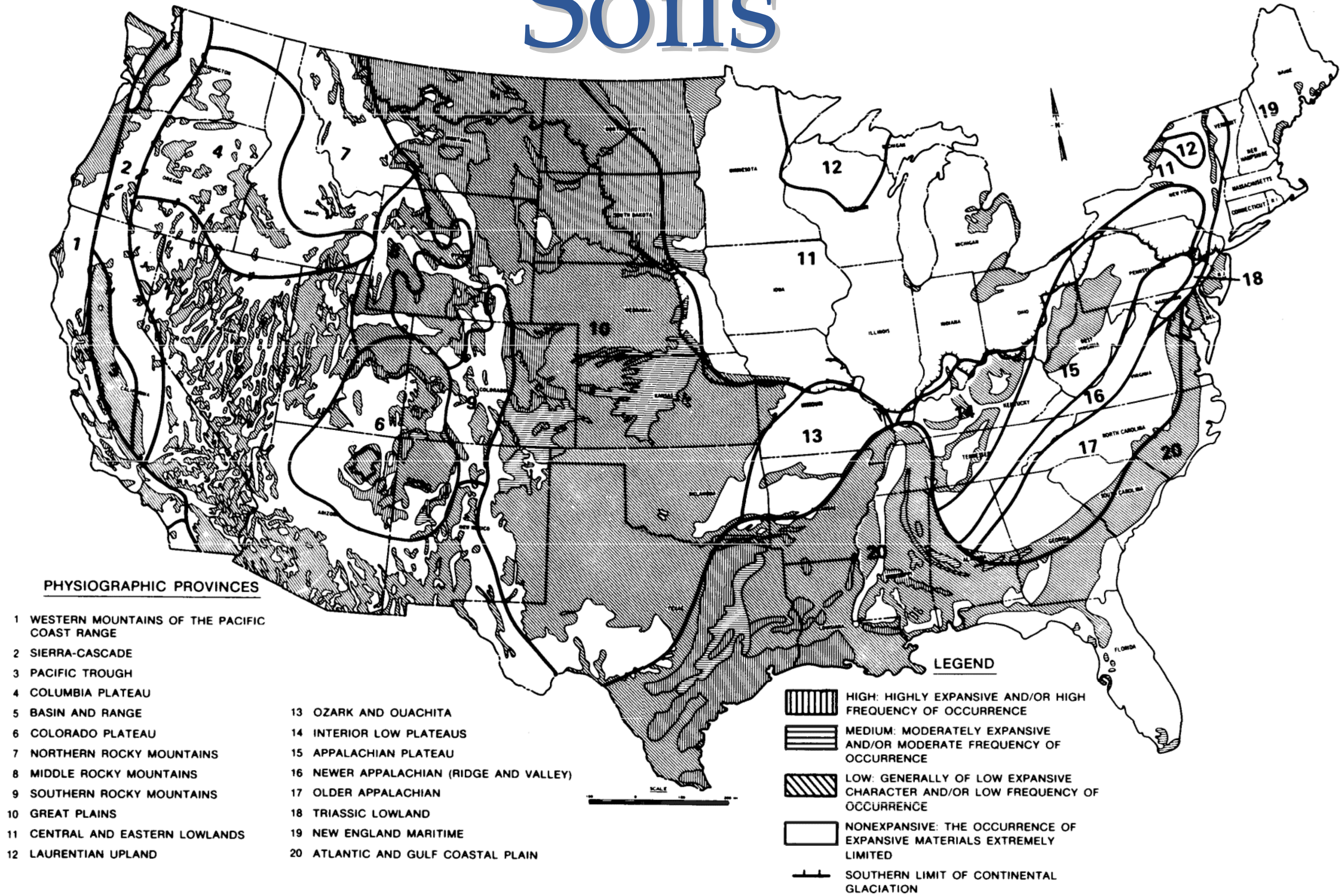
**Figure 5-5. Location of clay minerals on the Casagrande Plasticity Chart and Activity Index values (after Skempton, 1953, Mitchell, 1976, Holtz and Kovacs, 1981).**

**Modified Activity Index, $A_m$**: Based on their studies regarding the swell potential of compacted natural and artificial clay soils, Seed *et al.* (1962) proposed that for natural clay soils compacted as per the requirements of ASTM D 698 and Atterberg limits determined by ASTM D 4318 (AASHTO T 89, T 90), a Modified Activity Index, $A_m$, defined as follows is more appropriate:

$$A_m = \frac{PI}{CF - 5} \qquad 5\text{-}4$$

The above definition is used to define the swell potential of soils (see Section 5.7).

# Regions of Expansive Soils




U. S. Army Corps of Engineers

*Figure 2-1. Occurrence and distribution of potentially expansive materials in the United States, 1977, with boundaries of physiographic provinces.*

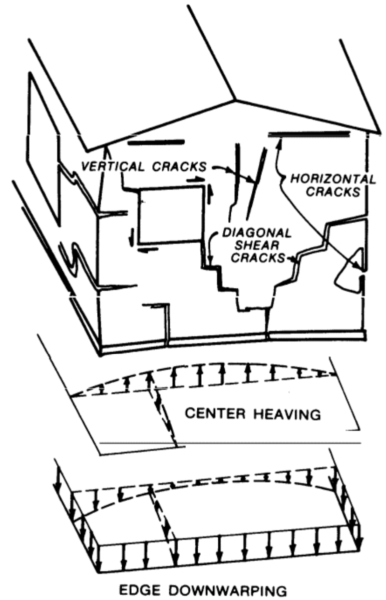
# Damage Due to Expansive Soils

a. Vertical cracks

b. Diagonal and vertical cracks

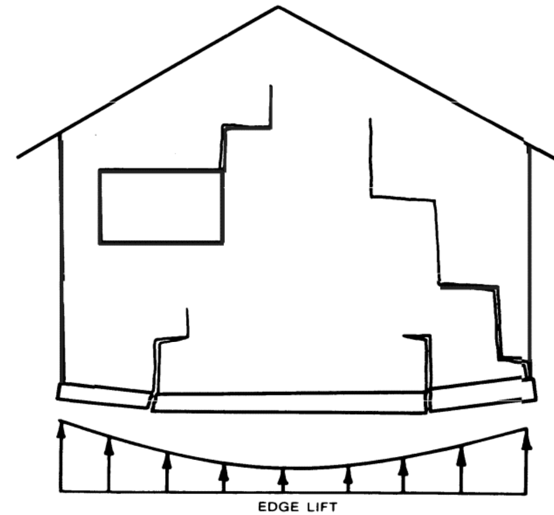
U. S. Army Corps of Engineers

*Figure 1-1. Examples of cracks in an exterior wall.*




U. S. Army Corps of Engineers

*Figure 1-2. Examples of wall fractures from doming heave of swelling and shrinking foundation soils.*




U. S. Army Corps of Engineers

*Figure 1-3. Examples of fractures from dish-shaped lift on swelling foundation soils.*

# Swell Potential

**5.7 VOLUME CHANGE PHENOMENA DUE TO LOADING AND MOISTURE**

Depending on mineralogy and depositional patterns, natural soils can exhibit either swell (expansion) or collapse under various degrees of loading and moisture ingress. Moisture may be in liquid or frozen form. For foundation design, it is very important to recognize and evaluate the potential for soils to swell or collapse. It is important to realize that these phenomena happen in both natural and compacted soils. Every year millions of dollars are spent dealing with the consequences of swelling (expanding) and collapsing soils. This section briefly discusses these two mechanisms and the tests that can be performed to evaluate the swell (expansion) and collapse potentials.

**5.7.1 Swell Potential of Clays**

Swelling is a characteristic reaction of some clays to water ingress. The potential for swell depends on the mineralogical composition of the soil fines. While montmorillonite (smectite) exhibits a high degree of swell potential, illite has no to moderate swell potential, and kaolinite exhibits almost none. The percentage of volumetric swell of a soil depends on the amount and type of clay, its relative density, the compaction moisture content and dry density, permeability, location of the groundwater table, the presence of vegetation and trees, overburden pressure, etc. Expansive soils are found throughout the U.S., however, damage caused by expansive clays is most prevalent in certain parts of California, Wyoming, Colorado, and Texas, where the climate is considered to be semi-arid and periods of intense rainfall are followed by long periods of drought. This pattern of wet and dry cycles results in periods of extensive near-surface drying and desiccation crack formation. During intense precipitation, water enters the deep cracks causing the soil to swell; upon drying, the soil will shrink. This weather pattern results in cycles of swelling and shrinking that can be detrimental to the performance of pavements, slabs on-grade, and retaining walls built on or in such soils.

Deep-seated volume changes in expansive soils are rare. More common are volume changes within the upper 3-10 feet (1- 3 m) of a soil deposit. These upper few feet are more likely to be affected by seasonal moisture content changes due to climatic changes. The zone over which volume changes are most likely to occur is defined as the active zone. The active zone can be evaluated by plotting the moisture content with depth for samples taken during the wet season and for samples taken during the dry season at the same location. The depth at which the moisture content becomes nearly constant is the limit of the active zone depth, which is also referred to as the depth of seasonal moisture change. The active zone is an important consideration in foundation design. In the design of piles or drilled shafts, it is important to recognize that full side friction resistance may not be realized in this zone. As the soil undergoes cycles of shrinking and swelling, it may lose contact with the pile or shaft. Alternatively, as the soil swells, it may impose significant uplift pressures on the foundation element.

In the field, the presence of surface desiccation cracks and/or fissures in a clay deposit is an indication of expansion potential. Experience has indicated that the most problematic expansive near-surface soils are typically highly plastic, stiff, fissured, overconsolidated clays. Several classification methods are used to identify expansive soils in the laboratory. Currently, there is not a standard classification procedure; different methods are used in various locations across the U.S. Typically, methods include the use of Atterberg limits and/or clay size percentage to describe a soil qualitatively as having low, medium, high, or very high expansion potential. Generally, soils with a plasticity index less than 15 percent will not exhibit expansive behavior. For soils with a plasticity index greater than 15 percent, the clay content of the soil should be evaluated in addition to the Atterberg limits. Figure 5-27 shows the swelling potential of natural soils and soils compacted to standard Proctor procedures (ASTM D 698) as a function of modified activity index, $A_m$, (Equation 5-4) and clay fraction.

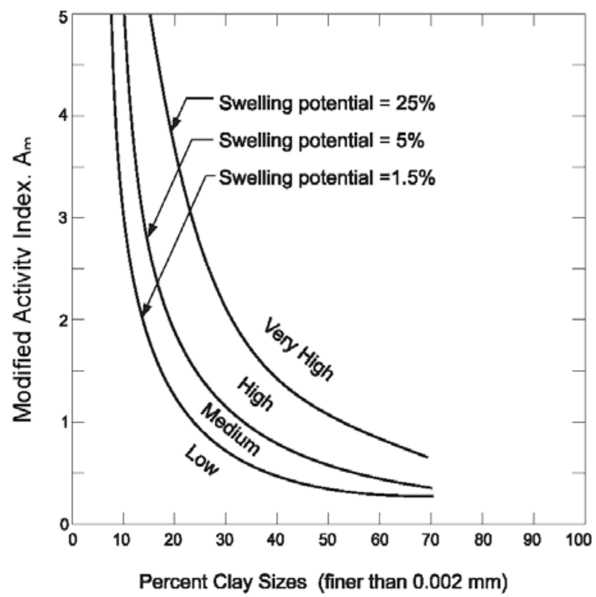



**Figure 5-27. Classification of swell potential for soils (after Seed *et al.*, 1962).**

# Swell Potential

**5.7.1.1 Evaluation of Expansion (Swell) Potential**

For situations where it is necessary to construct a facility in and around expansive soils, it will be necessary to estimate the magnitude of swell, i.e., surface heave, and the corresponding swelling pressures that may occur if the soil becomes wetted. The swelling pressure represents the magnitude of pressure that would be necessary to resist the tendency of the soil to swell. A one-dimensional swell potential test can be performed in an oedometer on undisturbed or recompacted samples according to AASHTO T256 or ASTM D 4546. In this test, the swell potential is evaluated by observing and measuring the swell of a laterally confined specimen when it is lightly surcharged and flooded with water. Alternatively, if the swelling pressure is to be measured, the height of the specimen is kept constant by adding load after the specimen is inundated. The swelling pressure is then defined as the vertical pressure necessary to maintain zero volume change. Swelling pressures in some expansive soils may be so large that the loads imposed by lightweight structures or pavements do little to counteract the swelling.

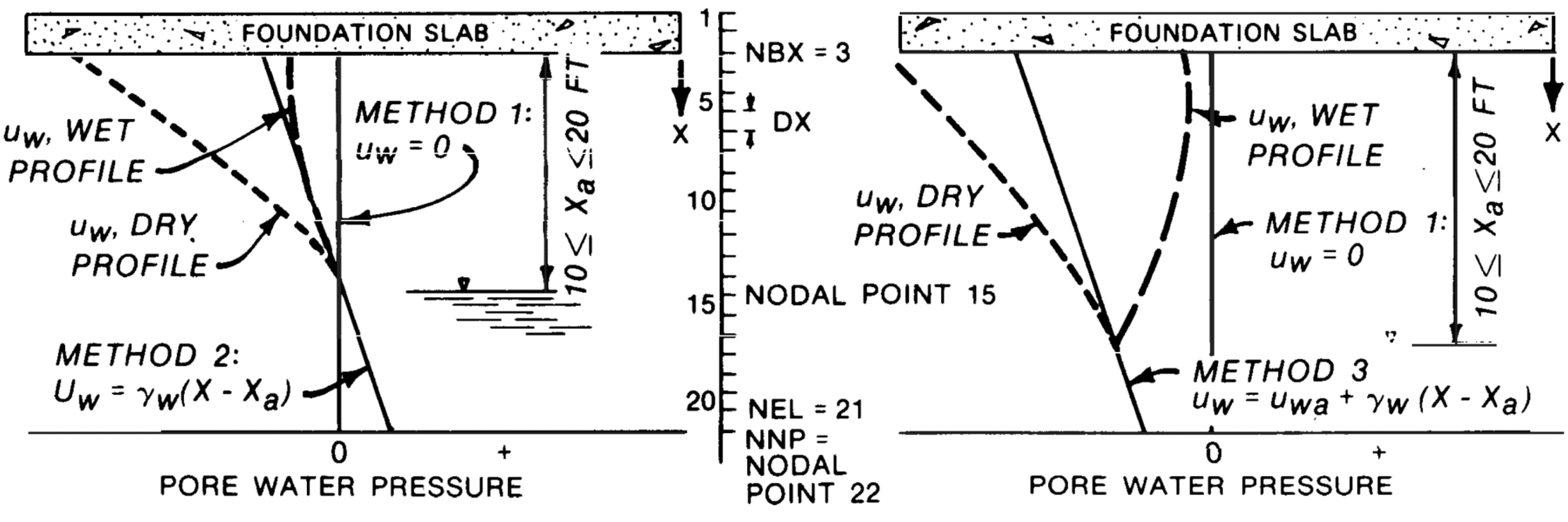
The use of the one-dimensional swell potential test to evaluate in-situ swell potential of natural and compacted clay soils has limitations including:

- Lateral swell and lateral confining pressure are not simulated in the laboratory. The calculated magnitude of swell in the vertical direction may not be a reliable estimate of soil expansion for structures that are not confined laterally (e.g., bridge abutments);

- The rate of swell calculated in the laboratory will not likely be indicative of the rate of swell experienced in the field. Laboratory tests cannot simulate the actual availability of water in the field.

It should be noted that there is a lack of a standard definition of swell potential in the technical literature based in part on variations in the test procedures, e.g., the condition of the test specimen (remolded or undisturbed), the magnitude of the surcharge, etc. Therefore the geotechnical specialist must be sure that the conditions used in the laboratory swell test simulate those expected in the field. In general, soils classified as CL or CH according to the USCS and A-6 or A-7 according to the AASHTO classification system should be considered potentially expansive.

# Determination of Actual Soil Expansion




a. Shallow Groundwater Level

b. Deep Groundwater Level

U. S. Army Corps of Engineers

*Figure 5-1. Assumed equilibrium pore water pressure profiles beneath foundation slabs.*

# Basis for Calculations

*a. Vertical movement.* Methodology for prediction of the potential total vertical heave requires an assumption of the amount of volume change that occurs in the vertical direction. The fraction of volumetric swell N that occurs as heave in the vertical direction depends on the soil fabric and anisotropy. Vertical heave of intact soil with few fissures may account for all of the volumetric swell such that N = 1, while vertical heave of heavily fissured and isotropic soil may be as low as N = 1/3 of the volumetric swell.

*b. Lateral movement.* Lateral movement is very important in the design of basements and retaining walls. The problem of lateral expansion against basement walls is best managed by minimizing soil volume change using procedures described in chapter 7. Otherwise, the basement wall should be designed to resist lateral earth pressures that approach those given by

$$\delta_h = K_o \delta_v \leqslant K_p \delta_v \qquad (5\text{-}1)$$

where

$\delta_h$ = horizontal earth pressure, tons per square root

$K_o$ = lateral coefficient of earth pressure at rest

$\delta_v$ = soil vertical or overburden pressure, tons per square foot

$K_p$ = coefficient of passive earth pressure

The $K_o$ that should be used to calculate $\delta_h$ is on the order of 1 to 2 in expansive soils and often no greater than 1.3 to 1.6.

*a. Basis of calculation.* The potential total vertical — heave at the bottom of the foundation, as shown in figure 5-1, is determined by

$$\Delta H = \mathbf{N \cdot DX} \sum_{i=NBX}^{i=NEL} \mathrm{DELTA}(i)$$

$$= \mathbf{N \cdot DX} \sum_{i=NBX}^{i=NEL} \frac{e_f(i) - e_o(i)}{1 + e_o(i)} \qquad (5\text{-}2)$$

where

AH = potential vertical heave at the bottom of the foundation, feet

N = fraction of volumetric swell that occurs as heave in the vertical direction

DX = increment of depth, feet

NEL = total number of elements

NBX = number of nodal point at bottom of the foundation

DELTA(i) = potential volumetric swell of soil element i, fraction

$e_f(i)$ = final void ratio of element i

$e_o(i)$ = initial void ratio of element i

The $\Delta H$ is the potential vertical heave beneath a flexible, unrestrained foundation. The bottom nodal point NNP = NEL + 1, and it is often set at the active depth **of heave $X_a$.**

# Van der Merwe Method

Van der Merwe's Method:

$$\Delta H = \sum_{i=1}^{n} \frac{(\Delta D)_i (F)_i}{\left(\bar{PE}\right)_i}$$

where

- $\Delta H$ =total change in height, feet or meters
- $i$ =layer index
- $n$ =total number of layers
- $\left(\bar{PE}\right)_i$ =inverse swell potential of layer i (see figure at right)
  - $= \infty$ (Low)
  - $= 48$ (Medium)
  - $= 24$ (High)
  - $= 12$ (Very High)
- $(\Delta D)_i$ =thickness of layer i, feet or meters
- $(F)_i$ =reduction factor for layer i
  - $= 10^{-\frac{D_i}{20}}$ , $D_i$ in feet
  - $= 10^{-\frac{D_i}{6}}$ , $D_i$ in meters
  - $D_i$ =depth of mid-point of layer from surface

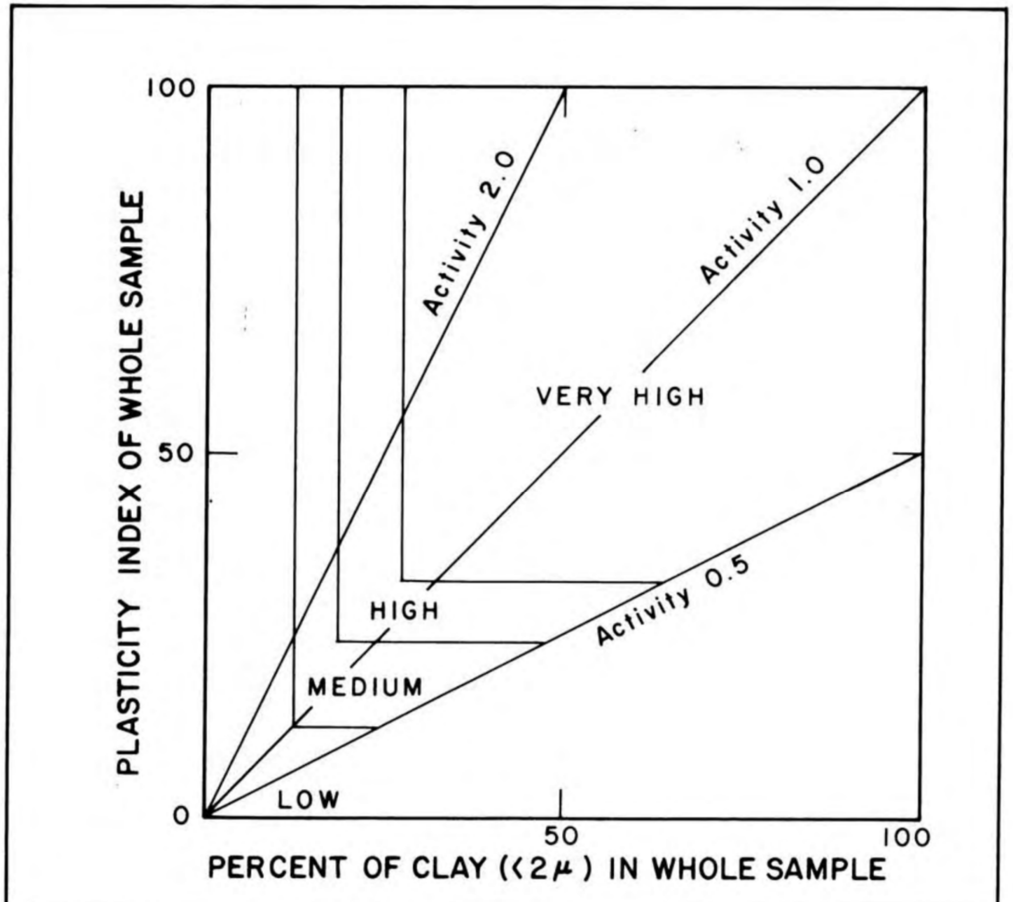



FIGURE 4
Volume Change Potential Classification for Clay Soils

# Example of Swell Calculation

- Given
  - Clay soil, PI = 40, CF = 55
- Find
  - Swell potential
  - Amount of vertical movement assuming only 1 m of soil is significant

- Solution
  - Compute Activity Indices
    - A = PI/CF = 40/55 = 0.72
    - Am = PI/(CF-5) = 40/50 = 0.8
  - Obtain swell potential
    - From Figure 5-27, High, approx. 10%
    - From van der Merwe's chart, Very High
  - Compute actual swell using van der Merwe's Method w/1 layer

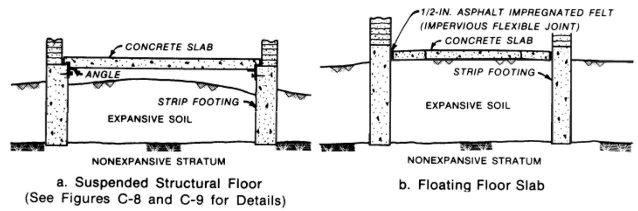
$$\Delta H = \frac{\Delta DF}{PE} = \frac{(1)\left(10^{-\frac{0.5}{6}}\right)}{12} = 0.069m = 69mm$$

# Foundations for Expansive Soils

*Table 6-1. Foundation Systems*

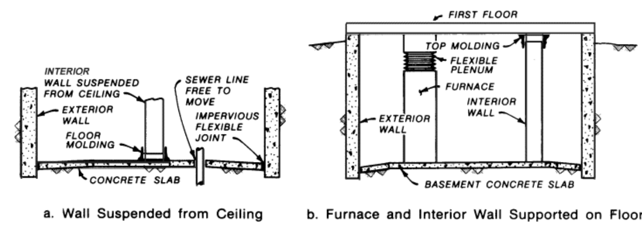
| Predicted Differential Movement, inches | Effective Plasticity Index, PI | Foundation System | Remarks | | |
|---|---|---|---|---|---|
| 1/2 | <15 | Shallow individual<br>Continuous wall<br>Strip | Lightly loaded buildings and residences. | | |
| | | Reinforced and stiffened thin mat | Residences and lightly loaded structures; on-grade 4- to 5-in. reinforced concrete slab with stiffening beams; maximum free area between beams 400 ft$^2$; 1/2 percent reinforcing steel; 10- to 12-in.-thick beams; external beams thickened or deepened, and extra steel stirrups added to tolerate high edge forces as needed; dimensions adjusted to resist loading. Beams positioned beneath corners to reduce slab distortion. | | |
| | | | **Type of Mat** | **Beam Depth, in.** | **Beam Spacing, ft** |
| 1/2 to 1 | 15 to 25 | | Light | 16 to 20 | 20 to 15 |
| 1 to 2 | 26 to 40 | | Medium | 20 to 25 | 15 to 12 |
| 2 to 4 | >41 | | Heavy | 25 to 30 | 15 to 12 |
| No limit | | Thick, reinforced mat | Large, heavy structures; mats usually 2 ft or more in thickness. | | |
| No limit | | Deep foundations, pile or drilled shaft | Foundations for any light or heavy structure; grade beams span between piles or shafts 6 to 12 in. above ground level; suspended floors or on-grade slabs isolated from grade beams and walls. Concrete drilled shafts may be underreamed or straight, reinforced, and cast in place with 3000-psi concrete of 6-in. slump. | | |

# Shallow Foundations for Expansive Soils




U. S. Army Corps of Engineers

*Figure 6–2. Footings on nonexpansive stratum.*




U. S. Army Corps of Engineers

*Figure 6–3. Interior joint details for slab-on-grade.*

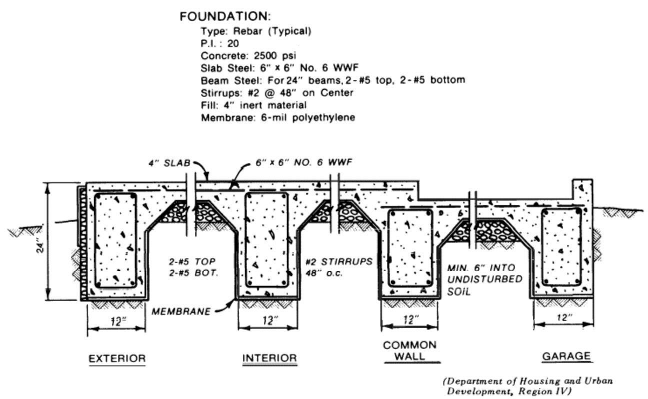



*Figure 6–5. Typical conventional rebar slab in Little Rock, Arkansas, for single-family, single-story, minimally loaded frame residence with 11- to 12-foot wall spacing.*

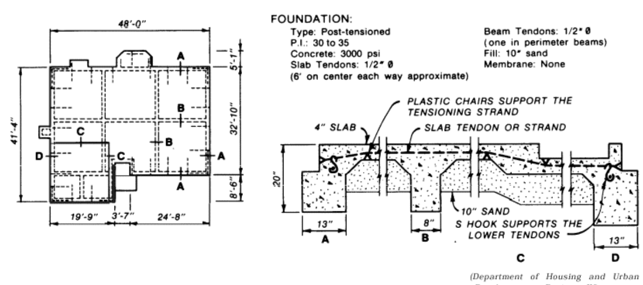
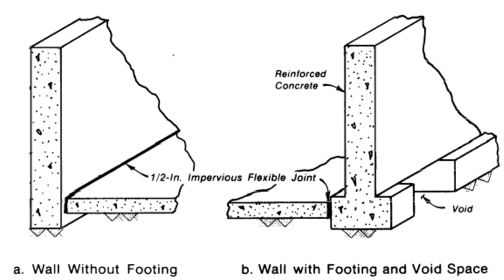


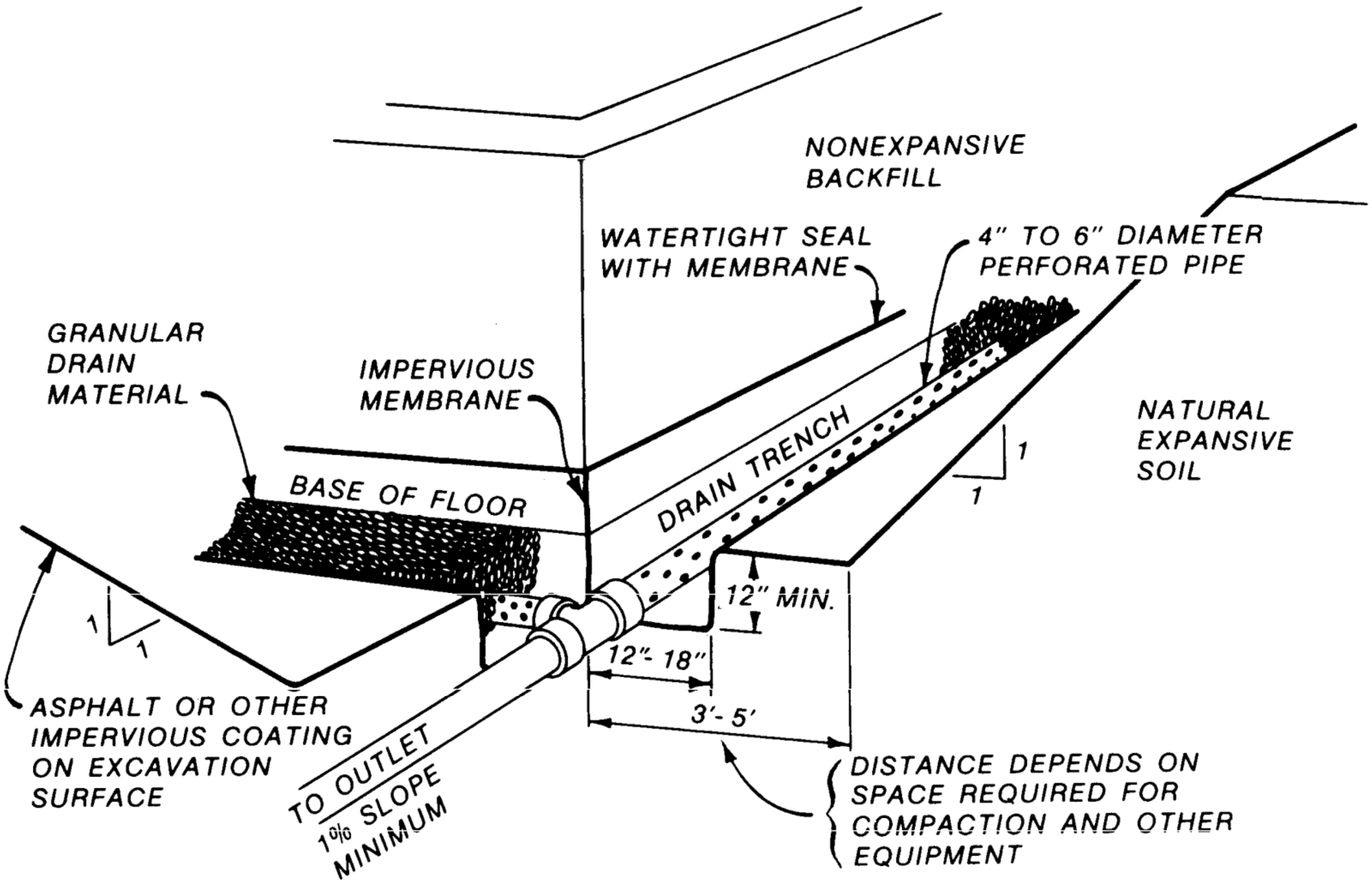

*Figure 6-6. Post-tensioned slab in Lubbock, Texas, for single-family, single-story, minimally loaded frame residence.*




U. S. Army Corps of Engineers

*Figure 6–4. Basement walls with slab-on-grade.*

# Use of Drainage Techniques




U. S. Army Corps of Engineers

*Figure 7-1. Drainage trench around outside of structure.*

# Upward Load Capacity of Belled Shafts

$$\left(P_{upward}\right)_a = \frac{\pi\left(B_b^2 - B_s^2\right)}{4} s_u N_u$$

$$N_u = 3.5\frac{D_b}{B_b} \le 9$$

(*Unfissured* Clays)

$$N_u = 0.7\frac{D_b}{B_b} \le 9$$

(*Fissured* Clays)

- Variables for uplift capacity
  - $(P_{upward})_a$ = ultimate/unfactored upward load capacity
  - $s_u$ = average undrained shear strength of the cohesive soil between the base of the bell and $2B_b$ above the base
  - $\sigma_{zD}$ = total stress at the bottom of the base
  - $B_b$ = diameter of the enlarged base
  - $B_s$ = diameter of the shaft
  - $D_b$ = depth of embedment of enlarged base into bearing stratum
- **Uplift of base only**

# Belled Bases for Drilled Shafts

- Use of an enlarged base, in conjunction with the shaft resistance of the straight portion, is a common way of dealing with expansive soils for major structures
- Enlarged bases offer additional uplift capacity
  - Only applicable to clays, since sands would usually collapse
  - Difficult to quantify

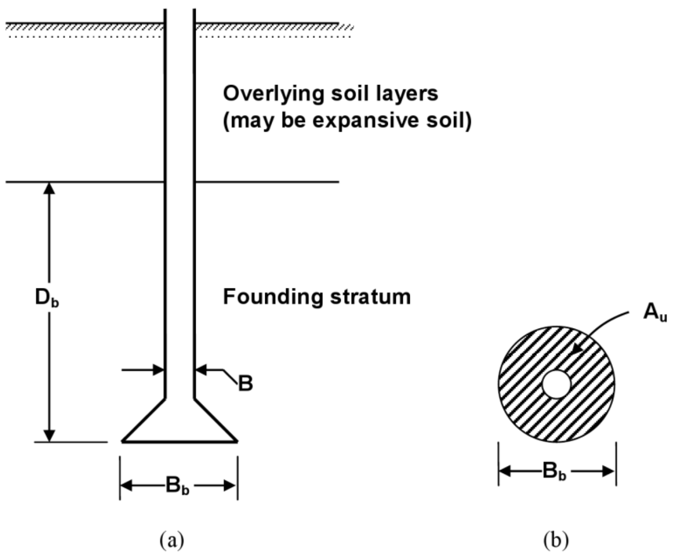
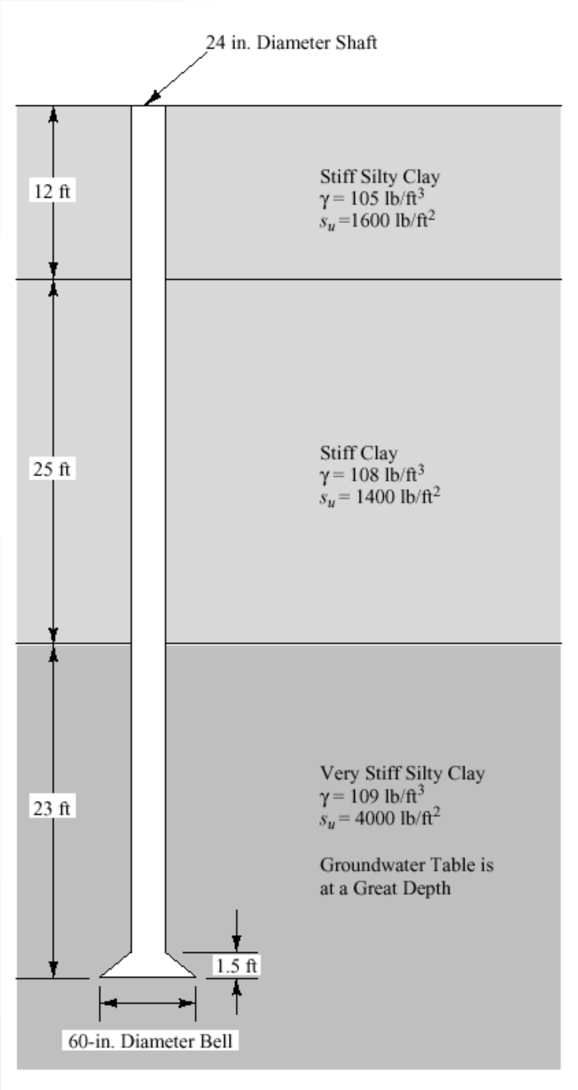



Figure C-10 (a) Geometry of Belled Shaft and (b) Projected Area of Bell in Uplift

# Example of Uplift Capacity




- Given
  - Drilled Shaft as shown
- Find
  - Uplift capacity of shaft
- Assume
  - Factor of safety = 5
  - Very stiff clay at toe is fissured
  - Include weight of foundation

# Compute Shaft Friction and $N_u$

| Layer | Layer Start | Layer End | Thickness, ft | su, psf | alpha | fs, psf | As, sq. ft |
|---|---|---|---|---|---|---|---|
| 1 | 0 | 5 | 5 | 1600 | 0 | 0 | 31.41593 |
| 2 | 5 | 12 | 7 | 1600 | 0.55 | 880 | 43.98 |
| 3 | 12 | 37 | 25 | 1400 | 0.55 | 770 | 157.08 |
| 4 | 37 | 50 | 13 | 4000 | 0.48 | 1920 | 81.68 |
| 5 | 50 | 60 | 10 | 4000 | 0 | 0 | 62.83 |
| | | | | | | Total | |

| | |
|---|---|
| Perimeter | 6.283185307 |
| Top Segment to be Ignored | 5 |
| Toe Segment to be Ignored | 10 |

Different from compression
Equal to twice the diameter of the belled toe

$$N_u = 0.7 \frac{D_b}{B_b} = 0.7 \frac{23}{5} = 3.22 \leq 9$$

# Compute Uplift Capacity

- Compute Ultimate Uplift Capacity of Bell

- Compute Weight of Foundation and Total Uplift Capacity

$$\left(P_{upward}\right)_a = \frac{\pi\left(B_b^2 - B_s^2\right)}{4} s_u N_u$$

$$\left(P_{upward}\right)_a = \frac{\pi\left(5_b^2 - 2_s^2\right)}{4}(4)(3.22)$$

$$\left(P_{upward}\right)_a = 212 \text{ kips}$$

$$W_f = \left(\frac{\pi 2^2}{4} 58.5 + \frac{\pi\left(\frac{2+5}{2}\right)^2}{4} 1.5\right) 0.150$$

$$W_f = 30 \ kips$$

$$\left(P_{upward}\right)_a = \frac{30 + 317 \times 0.75 + 212}{5} = 96 \ kips$$

# Questions