TM 5-818-7

TECHNICAL MANUAL

FOUNDATIONS IN EXPANSIVE SOILS

HEADQUARTERS, DEPARTMENT OF THE ARMY
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CHAPTER 5

METHODOLOGY FOR PREDICTION OF VOLUME CHANGES

5-1. Application of heave predictions

Reasonable estimates of the anticipated vertical and horizontal heave and the differential heave are necessary for the following applications.

a. Determination of adequate designs of structures that will accommodate the differential soil movement without undue distress (chap 6). These predictions are also needed to estimate upward drag from swelling soils on portions of deep foundations such as drilled shafts within the active zone of moisture change and heave. Estimates of upward drag help determine an optimum design of the deep foundation.

b. Determination of techniques to stabilize the foundation and to reduce the anticipated heave (chap 7).

5-2. Factors influencing heave

Table 5-1 describes factors that significantly influence the magnitude and rate of foundation movement. The difficulty of predicting potential heave is complicated beyond these factors by the effect of the type and geometry of foundation, depth of footing, and distribution of load exerted by the footing on the magnitude of the swelling of expansive foundation soil. Additional problems include estimating the exact location that swelling soils will heave or the point source of water seeping into the swelling soil and the final or equilibrium moisture profile in the areas of heaving soil.

Table 5-1. Factors Influencing Magnitude and Rate of Volume Change

Factor	Description
	Soil Properties
Composition of solids	A high percentage of active clay minerals include montmorillonites and mixed layer combinations of montmorillonites and other clay minerals that promote volume change.
Concentration of pore fluid salts	High concentrations of cations in the pore fluid tend to reduce the magnitude of volume change; swell from osmosis can be significant over long periods of time.
Composition of pore fluid	Prevalence of monovalent cations increase shrink-swell; divalent and trivalent cations inhibit shrink-swell.
Dry density	High initial dry densities result in closer particle spacings and larger swells.
Structure	Flocculated particles tend to swell more than dispersed particles; cemented particles tend to reduce swell; fabrics that slake readily may promote swell.
	Environmental Conditions
Climate	Arid climates promote desiccation, while humid climates promote wet soil profiles.
Groundwater	Fluctuating and shallow water tables (less than 20 ft from the ground surface) provide a source of moisture for heave.
Drainage	Poor surface drainage leads to moisture accumulations or ponding.
Vegetative cover	Trees, shrubs, and grasses are conducive to moisture depletion by transpiration; moisture tends to accumulate beneath areas denuded of vegetation.
Confinement	Larger confining pressures reduce swell; cut areas are more likely to swell than filled areas; lateral pressures may not equal vertical overburden pressures.
Field permeability	Fissures can significantly increase permeability and promote faster rates of swell.

5-3. Direction of soil movement

The foundation soil may expand both vertically and laterally. The vertical movement is usually of primary interest, for it is the differential vertical movement that causes most damages to overlying structures.

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

$$\sigma_h = K_o\sigma_v \leq K_p\sigma_v \qquad (5\text{-}1)$$

where

σ_h = horizontal earth pressure, tons per square root

K_o = lateral coefficient of earth pressure at rest

σ_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 σ_h is on the order of 1 to 2 in expansive soils and often no greater than 1.3 to 1.6.

5-4. potential total vertical heave

Although considerable effort has been made to develop methodology for reliable predictions within 20 percent of the maximum in situ heave, this degree of accuracy will probably not be consistently demonstrated, particularly in previously undeveloped and untested areas. A desirable reliability is that the predicted potential total vertical heave should not be less than 80 percent of the maximum in situ heave that will eventually occur but should not exceed the maximum in situ heave by more than 20 to 50 percent. Useful predictions of heave of this reliability can often be approached and can bound the in situ maximum levels of heave using the results of both consolidometer swell and soil suction tests described in paragraph 4-2a. The fraction N (para 5-3a) should be 1 for consolidometer swell test results and a minimum of 1/3 for soil suction test results. The soil suction tests tend to provide an upper estimate of the maximum in situ heave (N = 1) in part because the soil suction tests are performed without the horizontal restraint on soil swell that exists in the field and during one-dimensional consolidometer swell tests.

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

$$AH = N \cdot DX \sum_{i=NBX}^{i=NEL} DELTA(i)$$

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

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

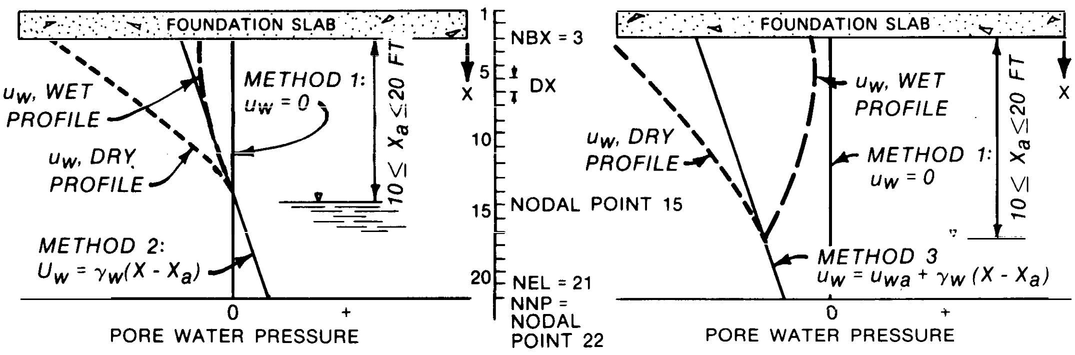
$e_o(i)$ = initial void ratio of element i

The AH 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.

(1) The initial void ratio, which depends on geologic and stress history (e.g., maximum past pressure), the soil properties, and environmental conditions shown in table 5-1 may be measured on undisturbed specimens using standard laboratory test procedures. It may also be measured during the laboratory swell tests as described in EM 1110-2-1906. The final void ratio depends on changes in the foundation conditions caused by construction of the structure.

(2) The effects of the field conditions listed in table 5-1 may be roughly simulated by a confinement pressure due to soil and structural loads and an assumption of a particular final or equilibrium pore water pressure profile within an active depth of heave X_a. The effects of confinement and the equilibrium pore water pressure profiles are related to the final void ratio by physical models. Two models based on results of consolidometer swell and soil suction tests are used in this manual (para 4-2a).

b. Pore water pressure profiles. The magnitude of swelling in expansive clay foundation soils depends on the magnitude of change from the initial to the equilibrium or final pore water pressure profile that will be observed to take place in a foundation soil because of



a. Shallow Groundwater Level

b. Deep Groundwater Level

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

the construction of the foundation.

(1) *Initial profile.* Figure 5-1 illustrates relative initial dry and wet profiles. The wet initial profile is probably appropriate following the wet season, which tends to occur by spring, while the dry initial profile tends to occur during late summer or early fall. The initial pore water pressure profile does not need to be known if the consolidometer swell model is used because the heave prediction is determined by the difference between the measured initial e_o and final e_1 void ratios (fig. 4-2). The initial void ratio is a function of the initial pore water pressure in the soil. The initial pore water pressure profile, which must be known if the soil suction model is used, may be found by the method described in appendix B.

(2) *Equilibrium profile. The* accuracy of the prediction of the potential total vertical heave in simulating the maximum in situ heave depends heavily on the ability to properly estimate the equilibrium pore water pressure profile. This profile is assumed to ultimately occur beneath the central portion of the foundation. The pore water pressure profile beneath the foundation perimeter will tend to cycle between dry and wet extremes depending on the field environment and availability of water. The three following assumptions are proposed to estimate the equilibrium profile. A fourth possibility, the assumption that the groundwater level rises to the ground surface, is most conservative and not normally recommended as being realistic. The equilibrium profile may also be estimated by a moisture diffusion analysis for steady-state flow, which was used to predict differential heave as part of the procedure developed by the Post-Tensioning Institute (PTI) for design and construction of slabs-on-grade (para 6-3b). The results, which should be roughly compatible with the hydrostatic profiles discussed in *(b)* and (c) below, lead to predictions of heave smaller than the saturated profile.

(a) Saturated. The saturated profile, Method 1 in figure 5-1, assumes that the in situ pore water pressure is zero within the active zone X_a of moisture change and heave

$$u_w = 0 \tag{5-3}$$

where u_w is the pore water pressure in tons per square foot at any depth X in feet within the active zone. Although a pore water pressure profile of zero is not in equilibrium, this profile is considered realistic for most practical cases and includes residences and buildings exposed to watering of perimeter vegetation and possible leaking underground water and sewer lines. Water may also condense in a layer of permeable subgrade soil beneath foundation slabs by transfer of water vapor from air flowing through the cooler subgrade. The accumulated water may penetrate into underlying expansive soil unless drained or protected by a moisture barrier. This profile should be used if' other information on the equilibrium pore water pressure profile is not available.

(b) Hydrostatic I. The hydrostatic I profile, Method 2 in figure 5-la, assumes that the pore water pressure becomes more negative with increasing vertical distance above the groundwater level in proportion to the unit weight of water

$$u_w = \gamma_w(X - X_a) \tag{5-4}$$

where γ_w is the unit weight of water (0.0312 ton per cubic foot).

This profile is believed to be more realistic beneath highways and pavements where drainage is good, pending of surface water is avoided, and leaking underground water lines are not present. This assumption will lead to smaller predictions of heave than the saturated profile of Method 1.

(c) *Hydrostatic II.* This profile, Method 3 in fig ure 5-lb, is similar to the hydrostatic I profile except that a shallow water table does not exist. The negative pore water pressure of this profile also becomes more negative with increasing vertical distance above the bottom of the active zone X_a in proportion to the unit weight of water

$$u_w = u_{wa} + \gamma_w(X - X_a) \quad (5\text{-}5)$$

where u_{wa} is the negative pore water pressure in tons per square foot at depth X_a in feet.

(d) Example application. Figure 5-2 illustrates how the saturated (Method 1) and hydrostatic II (Method 3) profiles appear for a suction profile without a shallow water table at a sampling site near Hayes, Kansas. The initial in situ soil suction or negative pore water pressure was calculated from the given natural soil suction without confining pressure τ_o by

$$\tau = \tau^{\circ} - \alpha\sigma_m \quad (5\text{-}6)$$

where

τ = in situ soil suction, tons per square foot
= compressibility factor (defined in app B)
σ_m = mean normal confining pressure, tons per square foot

The mean normal confining pressure σ_m is given by

$$\sigma_m = \frac{\sigma_v(1 + 2K_T)}{3} \quad (5\text{-}7)$$

where σ_v is the overburden or vertical confining pressure. The ratio of horizontal to vertical total stress K_T was assumed to be unity. The initial in situ soil suction τ was assumed to be essentially the matrix suction τ_m or negative pore water pressure u_w (i.e., the osmotic component of soil suction τ_s was negligible). The sign convention of the soil suction τ is positive, whereas that of the corresponding negative pore water pressure

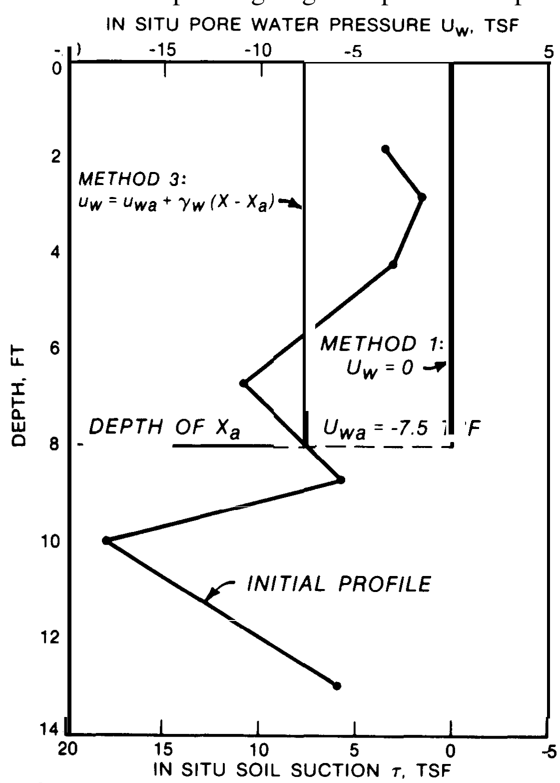

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Figure 5-2. Example application of equilibrium pore water pressure profile for a site near Hayes, Kansas.

u_w is negative (i.e., $\tau_m = -u_w$). Figure 5-2 shows that the hydrostatic equilibrium profile is nearly vertical with respect to the large magnitude of soil suction observed at this site. Heave will be predicted if the saturated profile occurs (Method 1 as in fig. 5-1), while shrinkage will likely be predicted if the hydrostatic II (Method 3) profile occurs. The availability of water to the foundation soil is noted to have an enormous impact on the volume change behavior of the soils. Therefore, the methods of chapter 7 should be used as much as practical to promote and maintain a constant moisture environment in the soil.

c. Depth of the active zone. The active zone depth X_a is defined as the least soil depth above which changes in water content and heave occur because of climate and environmental changes after construction of the foundation.

(1) *Shallow groundwater levels.* The depth X_a may be assumed equal to the depth of the water table for groundwater levels less than 20 feet in clay soil (fig. 5-1a). The u_{wa} term shown in figure 5-1b becomes zero for the hydrostatic I equilibrium profile in the presence of such a shallow water table.

(2) *Deep groundwater levels.* The depth X_a for deep groundwater levels may often be determined by evaluating the initial pore water pressure or suction with depth profile as described in appendix B, The magnitude of u., is then determined after the depth X_a is established.

(a) If depths to groundwater exceed 20 feet beneath the foundation and if no other information is available, the depth X_a can be assumed to be between 10 feet (for moist profiles or soil suctions less than 4 tons per square foot) and 20 feet (for dry profiles or soil suctions greater than 4 tons per square foot) below the base of the foundation (fig. 5-lb). However, the depth X_a should not be estimated less than three times the base diameter of a shaft foundation. Sources of moisture that can cause this active zone include the seepage of surface water down the soil-foundation interface, leaking underground water lines, and seepage from nearby new construction.

(b) The pore water pressure or soil suction is often approximately constant with increasing depth below X_a. Sometimes X_a can be estimated as the depth below which the water content/plastic limit ratio or soil suction is constant.

(c) If the soil suction is not approximately constant with increasing depth below depths of 10 to 20 feet, X_a may be approximated by being set to a depth 1 to 2 feet below the first major change in the magnitude of the soil suction, as shown in figure 5-2.

d. Edge effects. Predictions of seasonal variations in vertical heave from changes in moisture between extreme wet and dry moisture conditions (fig. 5-1) are for perimeter regions of shallow foundations. These calculations require a measure or estimate of both seasonal wet and dry pore water pressure or suction profiles. It should be noted from figure 5-lb that perimeter cyclic movement from extremes in climatic changes can exceed the long-term heave beneath the center of a structure.

(1) *Soil-slab displacements.* A slab constructed on the ground surface of a wet site may in time lead to downwarping at the edges after a long drought or growth of a large tree near the structure (fig. 5-3a). Edge uplift may occur following construction on an initially dry site (fig. 5-3b). The AH in figure 5-3 is representative of the maximum differential vertical heave beneath the slab, excluding effects of restraint from the slab stiffness, but does consider the slab weight.

(2) *Edge distance. The* edge lift-off distance e of lightly loaded thin slabs at the ground surface often varies from 2 to 6 feet but can reach 8 to 10 feet.

(3) *Deflection/length ratio. The* deflection/length ratio of the slab is A/L, where A is the slab deflection and L is the slab length. The angular deflection/span length Δ/ℓ (para 6-1*d*) is twice Δ/L of the slab (fig. 5-3).

e. Methods of predicting vertical heave.

(1) *Hand (manual) applications.* The heave ΔH from a consolidometer test may be found by

$$\frac{\Delta H}{H} = \frac{c_s}{1 + e_o} \log \frac{\sigma_s}{\sigma_v'} \qquad (5\text{-}8)$$

where

H = thickness of expansive soil layer, feet
c_s = swell index, slope of the curve between points 3 and 4, figure 4-2
σ_s = swell pressure, tons per square foot
σ_v' = final vertical effective pressure, tons per square foot

The final effective pressure is given by

$$\sigma_v' = \sigma_v - u_w \qquad (5\text{-}9)$$

where σ_v is the total vertical overburden pressure and u_w is the equilibrium pore water pressure in tons per square foot. If u_w is zero for a saturated profile, equation (5-3), then σ_v' is equal to σ_v and heave will be the same as that given by the equation for S_P in figure 4-2. A simple hand method and an example of predicting potential total vertical heave from consolidometer swell tests assuming a saturated equilibrium profile, equation (5-3), are given in TM 5-818-1 and in figure 5-4. However, hand calculations of potential heave can become laborious, particularly in heterogeneous profiles in which a variety of loading conditions need to be evaluated for several different designs,

(2) *Computer applications.* Predictions of potential total heave or settlement can be made quickly with the assistance of the computer program HEAVE available at the U. S. Army Corps of Engineers Waterways Experiment Station. The program HEAVE is applicable to slab, long continuous, and circular shaft foundations. This program considers effects of loading and soil overburden pressures on volume changes, heterogeneous soils, and saturated or hydrostatic equilibrium moisture profiles (equations (5-3) to (5-5)). Results of HEAVE using the saturated profile, equation (5-3), are comparable with results of manual computations described in figure 5-4.

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 and geometry of the structure also result in differential heave. Examples of effects of occupancy include broken or leaking water and sewer lines, watering of vegetation, and ponding adjacent to the structure. Other causes of differential heave include differences in the distribution of load and the size of footings.

a. *Unpredictability of variables.* Reliable predictions of future potential differential heave are often not possible because of many unpredictable variables that include: future availability of moisture from rainfall and other sources, uncertainty of the exact locations of heaving areas, and effects of human occupancy.

b. Magnitude of differential heave.

(1) Potential differential heave can vary from zero to as much as the total heave. Differential heave is often equal to the estimated total heave for structures supported on isolated spot footings or drilled shafts because some footings or portions of slab foundations often experience no movement. Eventually, differential heave will approach the total heave for most practical cases and should, therefore, be assumed equal to the total potential heave, unless local experience or other information dictates otherwise.

(2) The maximum differential heave beneath a lightly loaded foundation slab may also be estimated by the procedure based on the moisture diffusion theory and soil classification data developed by the PTI. Heave predictions by this method will tend to be less than by assuming that the differential heave is the total potential heave.

5-6. Heave with time

Predictions of heave with time are rarely reliable because the location and time when water is available to the soil cannot be readily foreseen. Local experience has shown that most heave (and the associated structural distress) occurs within 5 to 8 years following construction, but the effects of heave may also not be observed for many years until some change occurs in the

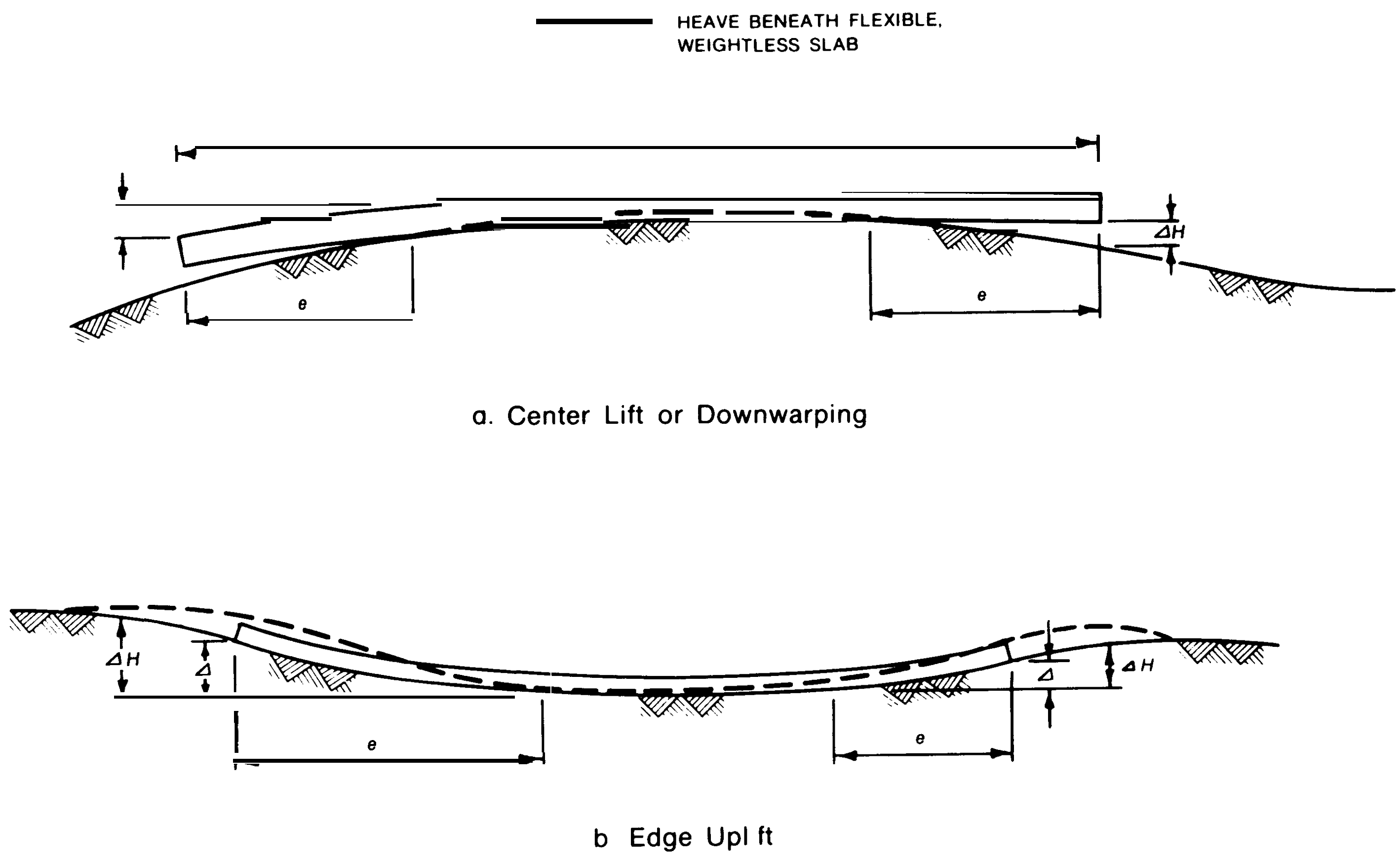

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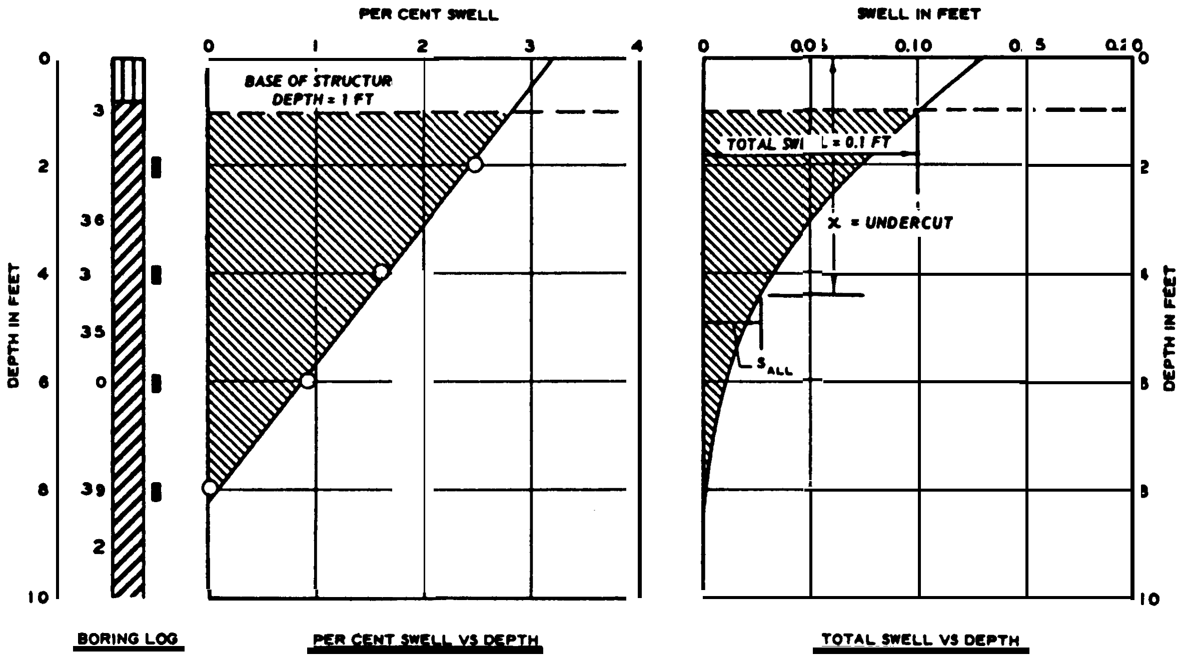
Figure 5-3. Soil-slab displacements on heaving soil.



NOTE: FIGURES TO LEFT OF BORING LOG ARE NATURAL WATER CONTENTS.

▮ DENOTES LOCATION OF SAMPLE FOR SWELL TESTS.

(A) PROCEDURE FOR ESTIMATING TOTAL SWELL

1. ON BASIS OF BORING LOG PROFILE SELECT SAMPLES AT INTERVALS FOR SWELL TESTS.
2. LOAD SPECIMENS IN CONSOLIDOMETER TO OVERBURDEN PRESSURE PLUS WEIGHT OF STRUCTURE; ADD WATER AND OBSERVE SWELL.
3. COMPUTE SWELL IN TERMS OF PER CENT OF ORIGINAL SPECIMEN HEIGHT AND PLOT VS DEPTH.
4. COMPUTE TOTAL SWELL WHICH IS EQUAL TO AREA ENCOMPASSED BY PER CENT SWELL VS DEPTH CURVE. FOR EXAMPLE, USING CURVES SHOWN ABOVE:

$$\text{TOTAL SWELL} = 1/2 \times (8.2 - 1.0) \times 2.8/100 = 0.10 \text{ FT}$$

(B) PROCEDURE FOR ESTIMATING AMOUNT OF UNDERCUT (x) NECESSARY TO REDUCE TOTAL SWELL TO AN ALLOWABLE VALUE (S_{ALL})

1. FROM PER CENT SWELL VS DEPTH RELATIONSHIP, COMPUTE AND PLOT TOTAL SWELL VS DEPTH RELATIONSHIP.
2. FOR A GIVEN VALUE OF S_{ALL}, THE AMOUNT OF UNDERCUT IS READ DIRECTLY OFF THE TOTAL SWELL-DEPTH CURVE.

NOTE: UNDERCUT MATERIAL SHOULD BE REPLACED BY INERT MATERIAL OR ELSE THE BASE OF THE STRUCTURE SHOULD BE LOWERED TO THE DEPTH OF THE REQUIRED UNDERCUT.

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Figure 5-4. Approximate method for computing foundation swell.

foundation conditions to disrupt the moisture regime. Predictions of when heave occurs are of little engineering significance for permanent structures. The important engineering problems are the determination of the magnitude of heave and the development of ways to minimize distress of the structure.

CHAPTER 6

DESIGN OF FOUNDATIONS

6-1. Basic considerations

a. Planning. Swelling of expansive foundation soils should be considered during the preliminary design phase and the level of structural cracking that will be acceptable to the user should be determined at this time.

(1) The foundation of the structure should be designed to eliminate unacceptable foundation and structural distress. The selected foundation should also be compatible with available building materials, construction skills, and construction equipment.

(2) The foundation should be designed and constructed to maintain or promote constant moisture in the foundation soils. For example, the foundation should be constructed following the wet season if possible. Drainage should be provided to eliminate ponded water. Excavations should be protected from drying. Chapter 7 describes the methods of minimizing soil movement.

b. Bearing capacity. Foundation loading pressures should exceed the soil swell pressures, if practical, but should be sufficiently less than the bearing capacity to maintain foundation displacements within tolerable amounts, Present theoretical concepts and empirical correlations permit reasonably reliable predictions of ultimate capacity, but not differential movement of the foundation. Factors of safety (FS) are therefore applied to the ultimate bearing capacity to determine safe or allowable working loads consistent with tolerable settlements. Further details on bearing capacity are presented in TM 5-818-1.

c. Foundation systems. An appropriate foundation should economically contribute to satisfying the functional requirements of the structure and minimize differential movement of the various parts of the structure that could cause damages. The foundation should be designed to transmit no more than the maximum tolerable distortion to the superstructure. The amount of distortion that can be tolerated depends on the design and purpose of the structure. Table 6-1 illustrates foundation systems for different ranges of differential movement or effective plasticity index ($\overline{PI}$) for proper selection of the foundation. Figure 6-1 explains the term $\overline{PI}$. The use of ΔH is preferred to $\overline{PI}$ because ΔH is a more reliable indicator of in situ heave. Also, $\overline{PI}$ is not a satisfactory basis of design in situations such as a 5-foot layer of highly swelling soil overlying nonswelling soil, rock, or sand. Pervious sand strata may provide a path for moisture flow into nearby swelling soil.

(1) *Shallow individual or continuous footings.* Shallow individual or long continuous footings are often used in low swelling soil areas where the predicted footing angular deflection/span length ratios are on the order of 1/600 to 1/1000 or 0.5 inch or less of movement.

(2) *Stiffened mats (slabs).* Stiffened mat foundations are applicable in swelling soil areas where predicted differential movement AH may reach 4 inches. The stiffening beams of these mats significantly reduce differential distortion. The range provided in table 6-1 for beam dimensions and spacings of stiffened slabs for light structures normally provides an adequate design.

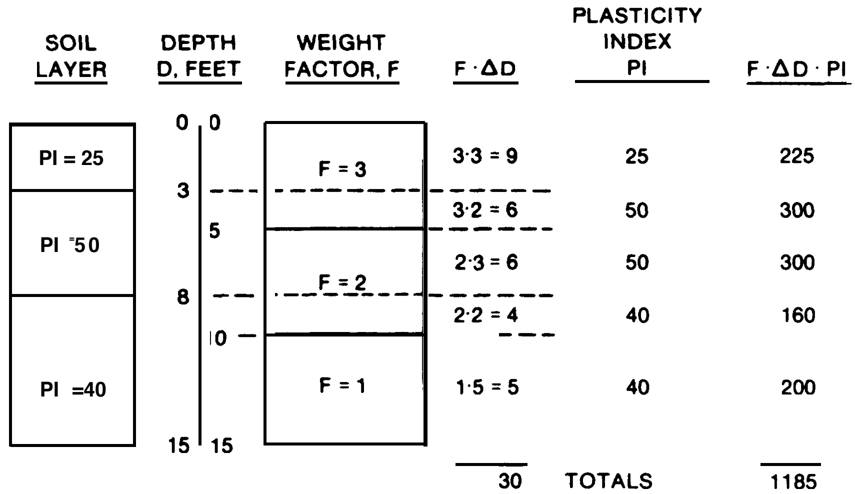
(3) *Deep foundations.* A pile or beam on a drilled shaft foundation is applicable to a large range of foundation soil conditions and tends to eliminate effects of heaving soil if properly designed and constructed (para 6-4). The type of superstructure and the differential soil movement are usually not limited with properly designed deep foundations. These foundations should lead to shaft deflection/spacing ratios of less than 1/600.

d. Superstructure systems. The superstructure should flex or deform compatibly with the foundation such that the structure continues to perform its functions, contributes aesthetically to the environment, and requires only minor maintenance. Frame construction, open floor plans, and truss roofs tend to minimize damage from differential movement. Load bearing walls tend to be more susceptible to damage from shear than the relatively flexible frame construction. Wood overhead beams of truss roof systems provide structural tension members and minimize lateral thrust on walls. Table 6-2 illustrates the relative flexibility provided by various superstructure systems.

(1) *Tolerable angular deflection/length ratios. The* ability of a structure to tolerate deformation depends on the brittleness of the building materials, length to height ratio, relative stiffness of the structure in shear and bending, and mode of deformation whether heave (dome-shaped, fig. 1-2) or settlement (dish-shaped, fig. 1-3). The vertical angular deflection/span length (Δ/ℓ) that can be tolerated, therefore, varies considerably between structures. The Δ/ℓ is the differential displacement Δ over the length ℓ between columns as

Table 6-1. Foundation Systems

Predicted Differential Movement, inches	Effective Plasticity Index, $\overline{PI}$	Foundation System	Remarks
1/2	<15	Shallow individual Continuous wall 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.



$\overline{PI}$ = 1185/30 = 39.4 or 40

Assumptions:

(1) The PI in the top and middle third is given 3 and 2 times as much weight (weight factor F), respectively, as the bottom one third to determine PI.

(2) A minimum PI of 15 should be used for any layer with PI less than 15.

(3) The PI should be increased by a slope factor F_s, in which log F_s = 0.01S; S = percent gradient in the slope of the ground surface.

(Based on data from Publication No. 1571, by the Building Research Advisory Council, 1968)

Figure 6-1. Effective plasticity index ($\overline{PI}$) or average $\overline{PI}$ in the top 15 feet of soil beneath the slab.

footings or about twice the A/L ratio of the slab (fig. 5-3). Only rough guidance of the range of tolerable Δ/ℓ ratios can be offered, such as in table 6-2, for different framing systems.

(a) Propagation of cracks depends on the degree of tensile restraint built into the structure and its foundation. Thus, frame buildings with panel walls are able to sustain larger relative deflections without severe damage than unreinforced load-bearing walls. Structural damage is generally less where the dish-shaped pattern develops than in the case of center heaving or edge downwarping because the foundation is usually better able to resist or respond to tension forces than the walls.

(b) A Δ/ℓ ratio of 1/500 is a common limit to avoid cracking in single and multistory structures. Plaster, masonry or precast concrete blocks, and brick walls will often show cracks for Δ/ℓ ratios between 1/600 to 1/1000. However, cracks may not appear in these walls if the rate of distortion is sufficiently slow to allow the foundation and frame to adjust to the new distortions. The use of soft bricks and lean mortar also tend to reduce cracking. Reinforced masonry, reinforced concrete walls and beams, and steel frames can tolerate Δ/ℓ ratios of 1/250 to 1/600 before cracks appear in the structure. Deflection ratios exceeding 1/250 are likely to be noticed in the structure and should usually be avoided. The Δ/ℓ ratios exceeding 1/150 usually lead to structural damage.

(2) *Provisions for flexibility.* The flexibility required to avoid undesirable distress may be provided by joints and flexible connections. Joints should be provided in walls as necessary, and walls should not be tied into the ceiling. Slabs-on-grade should not be tied into foundation walls and columns but isolated using expansion joints or gaps filled with a flexible, impervious compound. Construction items, such as reinforced concrete walls, stud frames, paneling, and gypsum board, are better able to resist distortions and should be used instead of brick, masonry blocks, or plaster walls. The foundation may be further reinforced by making the walls structural members capa-

Table 6-2. Superstructure Systems.

Superstructure system	Tolerable vertical angular deflection/ span length ratios, Δ/ℓ	Description
Rigid	1/600 to 1/1000	Precast concrete block, unreinforced brick, masonry or plaster walls, slab-on-grade.
Semirigid	1/360 to 1/600	Reinforced masonry or brick reinforced with horizontal and vertical tie bars or bands made of steel bars or reinforced concrete beams vertical reinforcement located on sides of doors and windows; slab-on-grade isolated from walls.
Flexible*	1/150 to 1/360	Steel, wood framing; brick veneer with articulated joints; metal, vinyl, or wood panels; gypsum board on metal or wood studs; vertically oriented construction joints; strip windows or metal panels separating rigid wall sections with 25-ft spacing or less to allow differential movement; all water pipes and drains into structure with flexible joints; suspended floor or slab-on-grade isolated from walls (heaving and cracking of slab-on-grade probable and accounted for in design).
Split construction*	1/150 to 1/360	Walls or rectangular sections heave as a unit (modular construction); joints at 25-ft spacing or less between units and in walls; suspended floor or slab-on-grade isolated from walls (probable cracking of slab-on-grade); all water pipes and drains equipped with flexible joints; construction joints in reinforced and stiffened slabs at 150-ft spacing or less and cold joints at 65-ft spacing or less.

* A Δ/ℓ value exceeding 1/250 is not recommended for normal practice, and a Δ/ℓ exceeding 1/150 often leads to structural damage.

ble of resisting bending such as reinforced concrete shear walls. Several examples of frame and wall construction are provided in appendix C.

6-2. Shallow individual or continuous footings

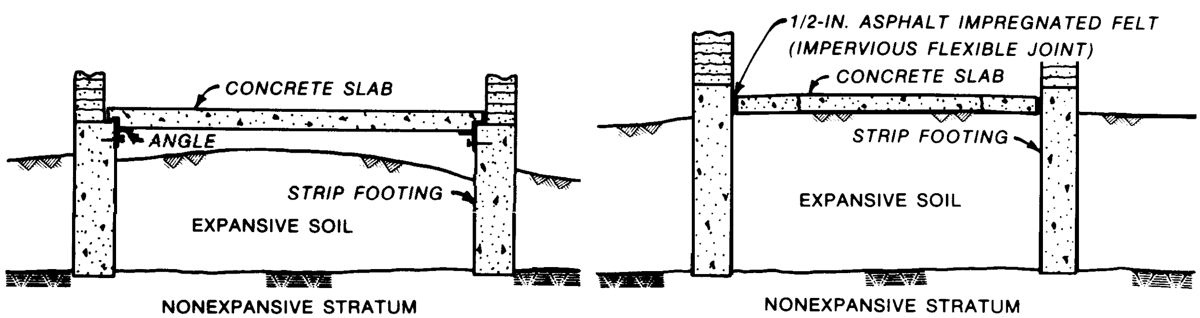
a. Susceptibility y to damage. Structures supported by shallow individual or continuous wall footings are susceptible to damages from lateral and vertical movement of foundation soil if provisions are not made to accommodate possible movement. Dishing or substantial settlement may occur in clays, especially in initially wet soil where a well-ventilated crawl space is constructed under the floor. The crawl space prevents rainfall from entering the soil, but the evaporation of moisture from the soil continues. Center heave or edge downwarping (fig. 1-2) can occur if the top layer of soil is permeable and site drainage is poor. Fractures may appear in walls not designed for differential **movement after Δ/ℓ ratios exceed 1/600 or differential** movement exceeds about 0.5 inch.

b. Applications. Shallow footings may be used where expansive strata are sufficiently thin to allow location of the footing in a nonexpansive or low-swelling stratum (fig. 6-2).

(1) A structural floor slab should be suspended on top of the footing (fig. 6-2a) or the slab-on-grade should be isolated from the walls (fig. 6-2b). The slab-on-grade should be expected to crack.

(2) Figure 6-3 illustrates examples of interior construction for a slab-on-grade. Interior walls may be suspended from the ceiling or supported on the floor. A flexible joint should be provided in the plenum between the furnace and the ceiling. Sewer lines and other utilities through the floor slab should be permitted to slip freely.

(3) Swelling of deep expansive soil beneath a nonexpansive stratum may cause differential movement of shallow footings if the moisture regime is altered in the deep soil following construction (e.g., change in groundwater level, or penetration of surface water into deep desiccated soil). Excavations for crawl spaces



a. Suspended Structural Floor
(See Figures C-8 and C-9 for Details)

b. Floating Floor Slab

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Figure 6-2. Footings on nonexpansive stratum.

or basements decrease the vertical confining pressure and pore water pressure, which can cause the underlying expansive foundation soil to heave from adjustment of the moisture regime back to the natural pore water pressures.

c. Basements. Basements and long continuous footings constructed in excavations are subject to swell pressures from underlying and adjacent expansive soil.

(1) *Walls.* Basement walls of reinforced concrete can be constructed directly on the foundation soil without footings provided foundation pressures are less than the allowable bearing capacity (fig. 6-4a). However, placing heavy loads on shallow footings may not be effective in countering high swell pressures because of the relative small width of the footings. The stress imposed on the soil is very low below a depth of about twice the width of the footing and contributes little to counter the swell pressure unless the expansive soil layer is thin.

(2) *Voids.* Voids can also be spaced at intervals beneath the walls to increase loading pressures on the foundation soil and to minimize flexing or bowing of the walls (fig. 6-4b). The voids may be made with removable wood forms, commercially available cardboard, or other retaining forms that deteriorate and collapse (para 6-4d).

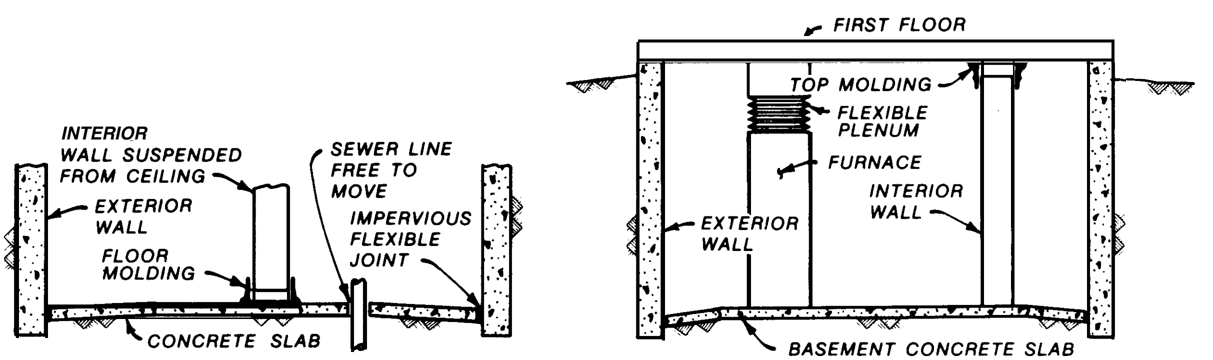
(3) *Joints.* Joints should be provided in interior walls and other interior construction if slab-on-ground is used (fig. 6-3). The slab should be isolated from the walls with a flexible impervious compound.

(4) *Lateral earth pressure on wall. The* coefficient of lateral earth pressure can exceed one if the backfill is heavily compacted and expansive, or the natural soil adjacent to the wall is expansive. Controlled backfills are recommended to minimize lateral pressures and increase the economy of the foundation (para 7-3a). Steel reinforcement can provide the necessary restraint to horizontal earth pressures, Unreinforced masonry brick and concrete blocks should not be used to construct basement walls.

d. Design. Standard design procedures for foundations of buildings and other structures are presented in TM5-818-1.

6-3. Reinforced slab-on-grade foundations

a. Application. The reinforced mat is often suitable for small and lightly loaded structures, particularly if



a. Wall Suspended from Ceiling

b. Furnace and Interior Wall Supported on Floor

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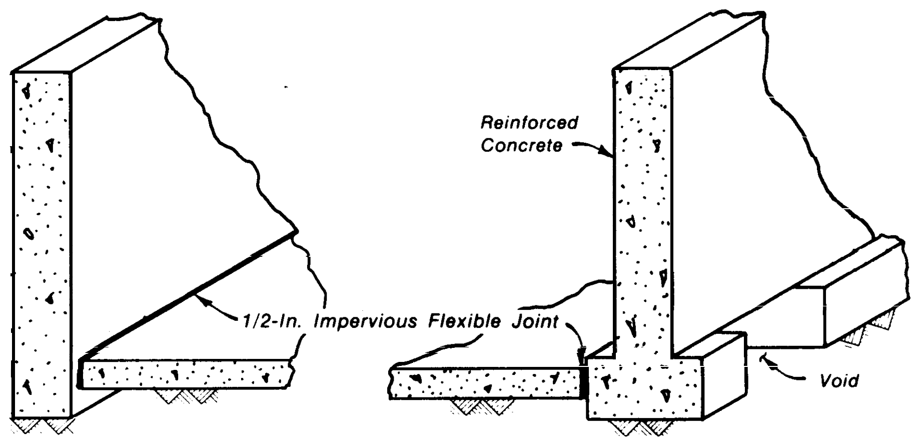
Figure 6-3. Interior joint details for slab-on-grade.



a. Wall Without Footing b. Wall with Footing and Void Space

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

the expansive or unstable soil extends nearly continuously from the ground surface to depths that exclude economical drilled shaft foundations. This mat is suitable for resisting subsoil heave from the wetting of deep desiccated soil, a changing water table, laterally discontinuous soil profiles, and downhill creep, which results from the combination of swelling soils and the presence of slopes greater than 5 degrees. A thick, reinforced mat is suitable for large, heavy structures. The rigidity of thick mats minimizes distortion of the superstructure from both horizontal and vertical movements of the foundation soil.

(1) *Effects of stiffening beams.* Concrete slabs without internal stiffening beams are much more susceptible to distortion or doming from heaving soil. Stiffening beams and the action of the attached superstructure with the mat as an indeterminate structure increase foundation stiffness and reduce differential movement. Edge stiffening beams beneath reinforced concrete slabs can also lessen soil moisture loss and reduce differential movement beneath the slab. However, the actual vertical soil pressures acting on stiffened slabs can become very nonuniform and cause localized consolidation of the foundation soil.

(2) *Placement of nonswelling layer.* Placement of a nonswelling, 6-inch-(or more) thick layer of (preferably) impervious soil on top of the original ground surface before construction of lightly loaded slabs is recommended to increase the surcharge load on the foundation soil, slightly reduce differential heave, and permit the grading of a slope around the structure leading down and away from it. This grading improves drainage and minimizes the possibility that the layer (if pervious) could be a conduit for moisture flow into desiccated foundation expansive soils. The layer should have some apparent cohesion to facilitate trench construction for the stiffening beams.

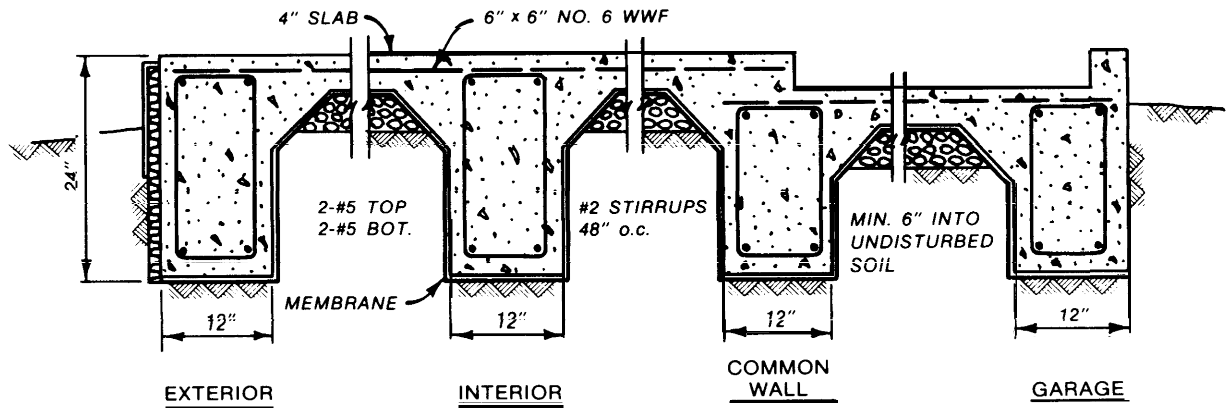
b. Design of thin slabs for light structures. Stiffened slabs may be either conventionally reinforced or posttensioned. The mat may be inverted (stiffening beams on top of the slab) in cases where bearing capacity of the surface soil is inadequate or a supported first floor is required. The Department of Housing and Urban Development, Region IV, San Antonio Area Office, has documented a series of successful conventionally reinforced and posttensioned slabs for the southern central states. Successful local practice should be consulted to help determine suitable designs.

(1) *Conventionally reinforced. The* conventional reinforced concrete waffle type mat (table 6-1), which is used for light structures, consists of 4- to 5-inch-thick concrete slab. This slab contains temperature steel and is stiffened with doubly reinforced concrete crossbeams. Figure 6-5 illustrates an engineered rebar slab built in Little Rock, Arkansas. Appendix C provides details of drawings of reinforced and stiffened thin mats. The 4-inch slab transmits the self-weight and first floor loading forces to the beams, which resist the moments and shears caused by differential heave of the expansive soil. Exterior walls, roof, and internal concentrated loads bear directly on the stiffening beams. Clearance between beams should be limited to 400 square feet or less. Beam spacings may be varied between the limits shown in table 6-1 to allow for concentrated and wall loads. Beam widths vary from 8 to 12 or 13 inches but are often limited to a minimum of 10 inches.

(a) *Concrete and reinforcement.* Concrete compressive strength f ‘c should be at least 2500 psi and preferably 3000 psi. Construction joints should be placed at intervals of less than 150 ft and cold joints less than 65 ft. About 0.5 percent reinforcing steel should be used in the mat to resist shrinkage and temperature effects.

FOUNDATION:

Type: Rebar (Typical)
P.I. : 20
Concrete: 2500 psi
Slab Steel: 6" x 6" No. 6 WWF
Beam Steel: For 24" beams, 2-#5 top, 2-#5 bottom
Stirrups: #2 @ 48" on Center
Fill: 4" inert material
Membrane: 6-mil polyethylene



(Department of Housing and Urban Development, Region IV)

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.

(b) Preliminary design, The three designs for reinforced and stiffened thin mats presented in table 6-1 differ in the beam depth and spacing depending on the predicted ΔH or $\overline{PI}$. The beam depths and spacings for each of the light, medium, and heavy slabs are designed for Δ/ℓ ratios of 1/500 and tend to be conservative in view of still undetermined fully acceptable design criteria and relatively high repair cost of reinforced and stiffened slabs. Stirrups may be added, particularly in the perimeter beams, to account for concentrated and exterior wall loads.

(2) *Post-tensioned.* Figure 6-6 illustrates an example of a posttensioned slab. Properly designed and constructed posttensioned mats are more resistant to fracture than an equivalent section of a conventional rebar slab and use less steel. However, post-tensioned slabs should still be designed with adequate stiffening beams to resist flexure or distortion from differential heave of the foundation soil, Experienced personnel are necessary to properly implement the posttensioning.

(3) *Assumptions of design parameters.* Design parameters include effects of climate, center and edge modes of differential swelling, perimeter and uniform loads, and structural dimensions.

(a) The effects of climate and differential swelling are accounted for by predictions of the maximum differential heave AH and the maximum edge lift-off distance e_m. Procedures for prediction of ΔH are provided in chapter 5. Reasonable values of the e_m are correlated with the Thornthwaite Moisture Index (TMI) in figure 6-7. The TMI, a climate related parameter roughly estimated from figure 6-8, represents the overall availability of water in the soil. The TMI can vary 10 to 20 or more (dimensionless) units from year to year. The e_m should be picked toward the top of the range in figure 6-7 for fissured soils. Since the e_m may exceed the range given in figure 6-7, depending on the activity of the soil or extreme changes in climatic conditions (e.g., long droughts and heavy rainfall), the value of e_m in feet may also be approximated by $2.5\Delta H$ with ΔH in inches for $\Delta H \leqslant 4$ inches.

(b) The loading distribution depends on the architectural arrangement of the building and often cannot be significantly altered. Perimeter and concentrated loads should be supported directly on the stiffening beams.

(c) The length and width of the slab are usually fixed by the functional requirement. Beam spacing depends on the slab geometry and varies between 10 and 20 feet. The depth of stiffening beams is controlled by the moment and shear capacity. The beam depth is adjusted as needed to remain within the allowable limits. The width of the stiffening beam is usually controlled by the excavation equipment and soil bearing capacity.

(4) *Structural design procedure,* The design proce-

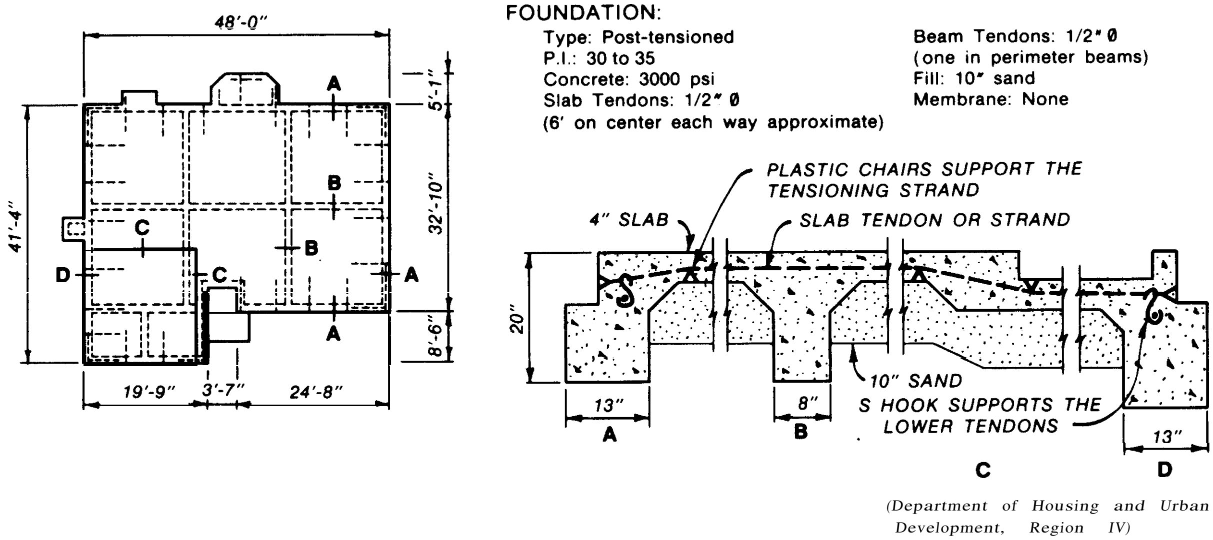


Figure 6-6. Post-tensioned slab in Lubbock, Texas, for single-family, single-story, minimally loaded frame residence.

dure should provide adequate resistance to shear, moment, and deflections from the structural loading forces, while overdesign is minimized. An economically competitive procedure that builds on the early work of the Building Research Advisory Board of the National Academy of Sciences is that developed for the Post-Tensioning Institute (PTI).

(a) The PTI procedure is applicable to both conventionally reinforced and posttensioned slabs up to 18 inches thick. It considers the previously discussed assumptions of the design parameters.

(*b*) The e_m and maximum differential heave y_m of the unloaded soil determined by the PTI procedure reflect average moisture conditions and may be exceeded if extreme changes in climate occur.

(c) Material parameters required by the PTI pro-

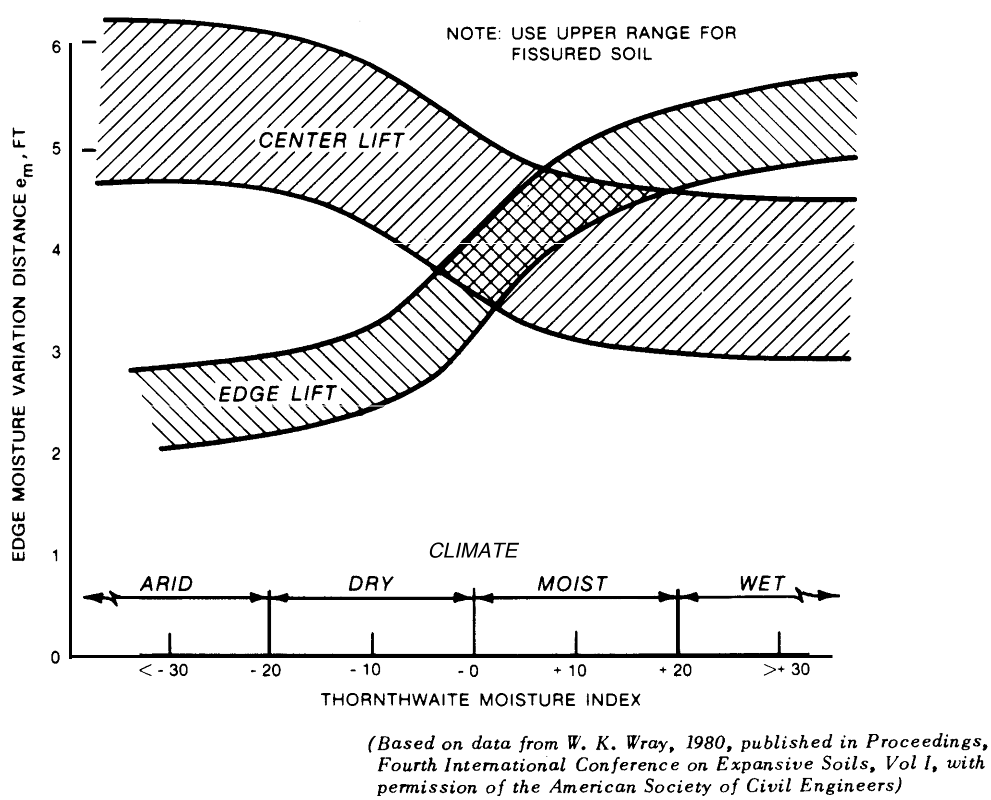


Figure 6-7. Approximate relationship between the Thornthwaite Moisture Index and the edge lift-off distance.

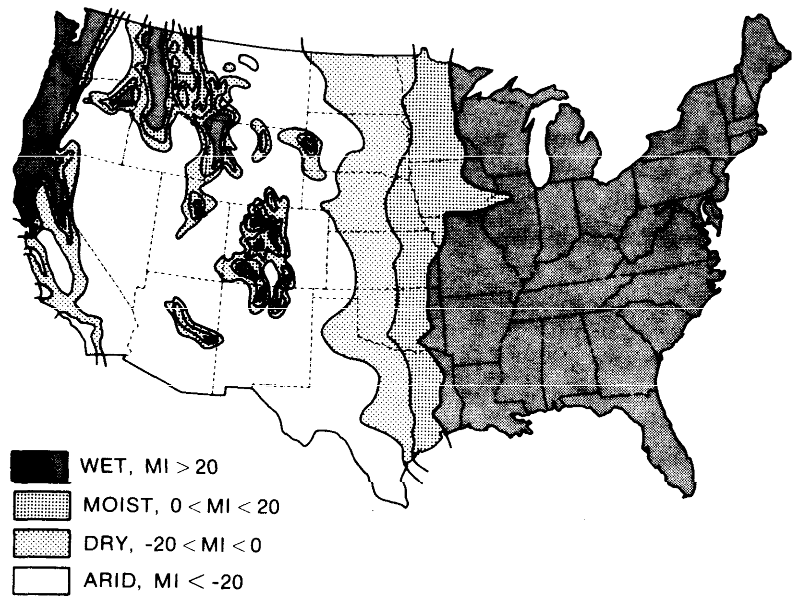


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Figure 6-8. Approximate distribution of the Thornthwaite Moisture Index (MI) in the United States.

cedure are the compressive strength of concrete; allowable tensile and compressive stresses in concrete; type, grade, and strength of the prestressing steel; grade and strength of the mild steel reinforcement; and slab subgrade friction coefficient, The amount of reinforcing steel recommended by this procedure should be considered a minimum. The slab-subgrade coefficient of friction should be 0.75 for concrete cast on polyethylene membranes and 1.00 if cast on-grade.

(*d*) **The allowable Δ/ℓ ratio must be estimated.** This ratio may be as large as 1/360 for center heave **and 1/800 for edge heave. The smaller edge Δ/ℓ ratio** criterion is recommended by the PTI because edge lift is usually much less than center lift deflections and the stems of the beams resisting the positive bending moment may be unreinforced.

c. Design of thick mats. The state of the art for estimating spatial variations in soil pressures on thick mats is often not adequate. These mats tend to be heavily overdesigned because of the uncertainty in the loading and the relatively small extra investment of some overdesign.

(1) *Description.* Concrete mats for heavy structures tend to be 3 feet or more in thickness with a continuous two-way reinforcement top and bottom. An 8-foot-thick mat supporting a 52-story structure in Houston, Texas, contains about 0.5 percent steel, while the 3-foot-thick mat of the Wilford Hall Hospital complex at Lackland Air Force Base in Texas also contains about 0.5 percent steel. The area of steel is 0.5 percent of the total area of the concrete distributed equally each way both top and bottom. The steel is overlapped near the concentrated loads, and a 3-inch cover is provided over the steel. The depth of the excavation that the mats are placed in to achieve bearing capacity and tolerable settlements eliminates seasonal edge effects such that the edge lift-off distance is not applicable.

(2) *Procedure.* The thick mat is designed to determine the shear, moment, and deflection behavior using conventional practice, then modified to accommodate swell pressures and differential heave caused by swelling soils. The analyses are usually performed by the structural engineer with input on allowable soil bearing pressures, uplift pressures (hydrostatic and swell pressures from expansive soils) and estimates of potential edge heave/shrinkage and center heave from the foundation engineer. Computer programs are commonly used to determine the shear, moments, and deflections of the thick mat.

(a) Structural solutions. The structural solution may be initiated with an estimate of the thickness of a spread footing that resists punching shear and bending moments for a given column load, concrete compressive strength, and soil bearing capacity. Following an estimation of the initial thickness, hand solutions of mat foundations for limited application based on theory of beams on elastic foundations are available from NAVFAC DM-7. More versatile solutions are available from computer programs based on theory of beams on elastic foundations such as BMCOL 2, which is available at the U.S. Army Corps of Engineer Waterways Experiment Station, and finite element analysis.

(b) Foundation soil/structure solutions. The BMCOL 2 soil-structure interaction program permits nonlinear soil behavior. Finite element programs are

also available, but they are often burdened with hard to explain local discontinuities in results, time-consuming programming of input data, and need of experienced personnel to operate the program. The finite element program originally developed for analysis of Port Allen and Old River Locks was applied to the analysis of the Wilford Hall Hospital mat foundation at Lackland Air Force Base in Texas. Figure 6-9 compares predicted with observed movement of the 3.5-foot-thick mat at Wilford Hall. Foundation soils include the fissured, expansive Navarro and upper Midway clay shales. These computer programs help refine the design of the mat and can lead to further cost reductions in the foundation.

6-4. Deep foundations

The deep foundation provides an economical method for transfer of structural loads beyond (or below) unstable (weak, compressible, and expansive) to deeper stable (firm, incompressible, and nonswelling) strata. Usually, the deep foundation is a form of a pile foundation. Numerous types of pile foundations exist of which the most common forms are given in table 6-3. Occasionally when the firm-bearing stratum is too deep for the pile to bear directly on a stable stratum, the foundation is designed as friction or floating piles and supported entirely from adhesion with the surrounding soil and/or end bearing on underreamed footings.

a. General applications. Each of the types of piling is appropriate depending on the location and type of structure, ground conditions (see table 3-1 for examples), and durability. The displacement pile is usually appropriate for marine structures. Any of the piles in table 6-3 may be considered for land applications. Of these types the bored and cast in situ concrete drilled shaft is generally more economical to construct than driven piles.

b. Application of drilled shafts. Table 6-4 describes detailed applications of drilled shaft foundations including advantages and disadvantages. Detailed discussion of drilled shaft foundations is presented below because these have been most applicable to the solution of foundation design and construction on expansive clay soils.

(1) A drilled shaft foundation maybe preferred to a mat foundation if excavating toward an adequate bearing stratum is difficult or the excavation causes settlement or loss of ground of adjacent property.

(2) A drilled shaft foundation 20 to 25 feet deep tends to be economically competitive with a ribbed mat foundation,

(3) Drilled shafts may be preferred to mat foundations if differential heave ΔH exceeds 4 inches or Δ/ℓ ratios exceed 1/250, Mat foundations under such conditions may tilt excessively leading to intolerable distortion or cracking.

(4) The shaft foundation may be economical com-

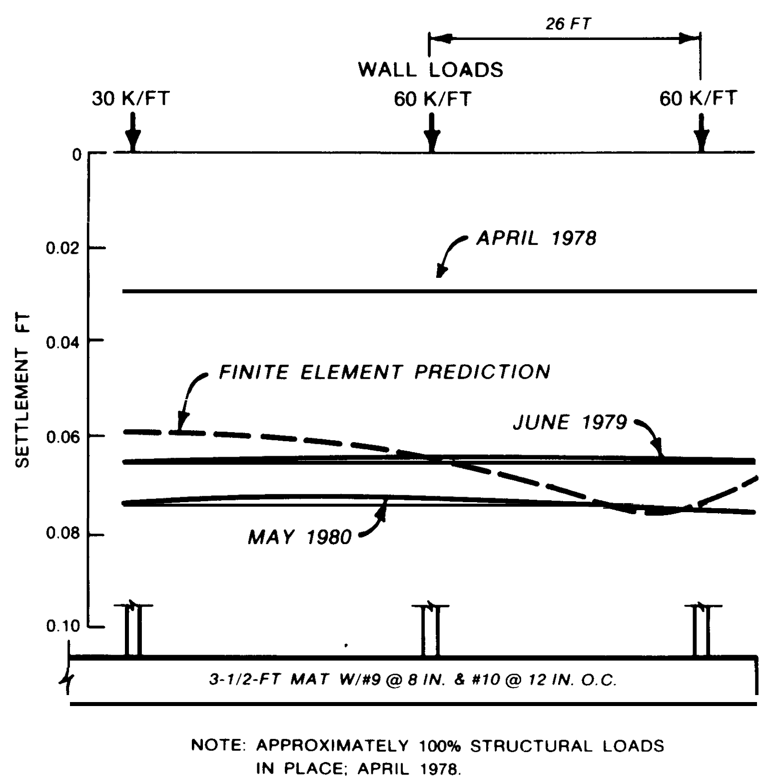

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Figure 6-9. Settlement and deflection of a mat foundation.

Table 6-3. Classification of Piles

Classification	Type	Description
Displacement	Timber Precast concrete Steel circular or rectangular Tapered timber or steel	Driven piles with solid circular or rectangular cross section or hollow section with closed bottom end. Piles hammered or jacked down into place.
Small displacement	Precast concrete Prestressed concrete Steel H section Steel circular or rectangular Screw	Small cross-section pile consisting of open-end cylinder, rectangular, H section, or screw configuration.
Nondisplacement	Drilled shaft Tubes filled with concrete Precast concrete Injected cement mortar Steel section	Piles placed in open boreholes. Usually concrete placed in holes drilled by rotary auger, baling, grabbing, airlift, or reverse circulation methods.
Combination	Steel-driven tube replaced by concrete Precast shell filled with concrete Jointed pile of different materials	Combination of different forms of piles.

pared with traditional strip footings, particularly in open construction areas and with shaft lengths less than 10 to 13 feet, or if the active zone is deep, such as within areas influenced by tree roots.

c. *General considerations.*

(1) *Causes of distress.* The design and construction of drilled shaft foundations must be closely controlled to avoid distress and damage. Most problems have been caused by defects in construction and by inadequate design considerations for effects of swelling soil (table 6-5). The defects attributed to construction techniques are discontinuities in the shaft, which may occur from the segregation of concrete, failure to complete concreting before the concrete sets, and early set of concrete, caving of soils, and distortion of the steel reinforcement. The distress resulting from inadequate design considerations are usually caused by wetting of subsoil beneath the base, uplift forces, lack of an air gap beneath grade beams, and lateral movement from downhill creep of expansive clay.

(2) *Location of base.* The base of shafts should be located below the depth of the active zone, such as below the groundwater level and within nonexpansive soil. The base should not normally be located within three base diameters of an underlying unstable stratum.

(a) Slabs-on-grade isolated from grade beams and walls are often used in light structures, such as residences and warehouses, rather than the more costly structural slabs supported by grade beams and shafts. These slabs-on-grade will move with the expansive soil and should be expected to crack.

(b) To avoid "fall-in" of material from the granular stratum during underreaming of a bell, the base may be placed beneath swelling soil near the top of a granular stratum.

(3) *Underreams.* Underreams are often used to increase anchorage to resist uplift forces (fig. 6-10). The belled diameter is usually 2 to 2.5 times the shaft diameter D_s and should not exceed 3 times D_s. Either 45- or 60-degree bells may be used, but the 45-degree bell is often preferred because concrete and construction time requirements are less. Although the 45-degree bell may be slightly weaker than the 60-degree bell, no difference has been observed in practice. The following considerations are necessary in comparing underreamed shafts with straight shafts.

(a) Straight shafts may be more economical than underreams if the bearing stratum is hard or if subsoils are fissured and friable. Soil above the underream may be loose and increase the upward movement needed to develop the bell resistance.

(b) The shaft can often be lengthened to eliminate the need for an underream, particularly in soils where underreams are very difficult to construct. Friction resistance increases rapidly in comparison with end bearing resistance as a function of the relative shaft-soil vertical movement.

(c) Underreams reduce the contact bearing pressure on potentially expansive soil and restrict the minimum diameter that may be used.

(4) *Uplift forces.* If bells or underreams are not feasible, uplift forces (table 6-5) may be controlled by the following methods:

(a) The shaft diameter should be the minimum required for downloads and construction procedures and control.

Table 6-4. Applications of Drilled Shafts

Applications	Advantages	Disadvantages
Absence of a shallow, stable founding stratum; support of structures with shafts drilled through swelling soils into zones unaffected by moisture changes.	Personnel, equipment, and materials for construction usually readily available; rapid construction due to mobile equipment; careful inspection of excavated hole usually possible; noise level of equipment less than some other construction methods; low headroom needed.	Accurate predictions of load and settlement behavior not always possible.
Support of moderate-to-high column loads; high column loads with shafts drilled into hard bedrock; moderate column loads with underreamed shafts bottomed on sand and gravel.	Excavated soil examined to check the projected soil conditions and profile; excavation possible for a wide variety of soil conditions.	Careful design and construction required to avoid defective foundations; careful inspection necessary during construction; inspection of concrete after placement difficult.
Support of light structures on friction shafts.	Heave and settlement at the ground surface normally small for properly designed shafts.	Inadequate knowledge of design methods and construction problems leading to improper design; strict requirements for investigations.
Rigid limitations on allowed structure deformations at site where differential heave or settlement is predicted to exceed 3 to 4 in.; large lateral variations in soil conditions.	Disturbance of soil minimized by drilling, thus reducing consolidation and dragdown due to remolding compared to other methods of placing deep foundations such as driving.	Construction techniques sometimes very sensitive to subsurface conditions: susceptible to "necking" in squeezing ground; difficult to concrete requiring tremie if hole filled with slurry or water; cement washing out if water is under artesian pressure; pulling casing disrupting continuity of concrete in the shaft or displacing/distorting the reinforcing cage.
Structural configurations and functional requirements or economics precluding a mat or other foundation; resisting uplift forces from swelling soils; and providing anchorage to pulling, lateral, or overturning forces.	A single shaft carrying very large loads.	Heave beneath base of shaft aggravating movement beneath slab-on-grade.
	Pile caps eliminated.	Failures difficult and expensive to correct.
	Changes in geometry (diameter, penetration, underream) made during construction if required by subsurface conditions.	

Table 6-5. Defects Associated with Drilled Shafts

Defect	Remarks
Defects from Construction Techniques	
Discontinuities in the shaft	Do not leave cuttings in the borehole prior to concreting. Too rapid pulling of casing can cause voids in the concrete. Avoid ground-water pressure greater than concrete pressure, inadequate spacing in steel reinforcement, and inadequate concrete slump and workability.
Reduced diameter from caving soil	Caving or squeezing occurs along the shaft in cohesionless silt, rock flour, sand or gravel, and soft soils, especially below the water table. Coarse sands and gravels cave extensively during drilling and tend to freeze casing in place. Soft soils tend to close open boreholes. Raising the auger in soft soils may "suck" the borehole to almost complete closure.
Distortion of reinforcement	Distortion of steel reinforcement cages can occur while the casing is pulled. Horizontal bands or ties should be placed around reinforcing steel.
Defects Attributed to Swelling Soil	
Mode of Defect	**Remarks**
Subsoil wetting below base of shaft	Moisture may migrate down the concrete of the shaft from the surface or from perched water tables into deeper desiccated zones, causing the entire shaft to rise. Shafts may also heave from a rising deep water table. Rise is sometimes avoided by increasing the shaft length or placing the base in nonswelling soil or within a water table.
Uplift	Heave of surrounding desiccated swelling clays can cause friction forces, which in time cause the shaft to rise and even fracture from excessive tensile stress. Rise can be reduced by placing an underreamed base in nonswelling soil, increasing steel reinforcement along the entire shaft length and underreamed base to resist the tensile stress, and providing sleeving to reduce adhesion between the shaft and soil.
Grade beams on swelling soil	Lack of an air gap between the surface of swelling soil and the grade beam can cause the grade beam to rise.
Lateral swell	Shaft foundations have low resistance to damage from lateral swell. Downhill creep of expansive clays contributes to damaged foundations.

(b) The shaft length may be extended further into stable, nonswelling soil to depths of twice the depth of the active zone X_a.

(c) Widely spaced shafts may be constructed with small diameters and a total loading force Q_w that exceeds the maximum uplift thrust (fig. 6-11) expressed as

$$Q_u = \pi D_s \int_0^{L_n} f_s dL < Q_w \qquad (6\text{-}1)$$

where

Q_u = maximum uplift thrust on perimeter of shaft, tons

D_s = diameter of shaft, feet

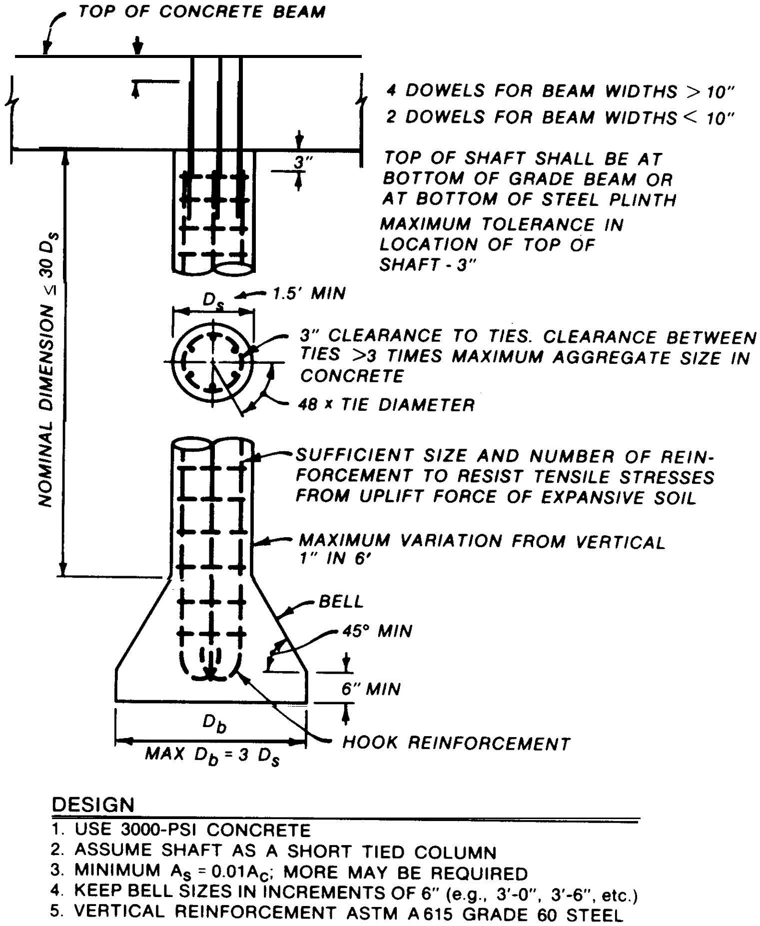
L_n = thickness of the swelling layer moving up relative to the shaft, feet

f_s = skin resistance, tons per square foot

dL = differential increment of shaft length L, feet

The point n in figure 6-11 is the neutral point. The value of L_n should be approximately equal to the depth X_a. The maximum skin resistance f_s is evaluated in *d* below. The loading force Q_w should also be less than or equal to the soil allowable bearing capacity. Wide spans between shafts also reduce angular rotation of the structural members. The minimum spacing of shafts should be 12 feet or 8 times the shaft diameter (whichever is smaller) to minimize effects of adjacent shafts.

(d) The upper portion of the shaft should be kept vertically plumb (maximum variation of 1 inch in 6 feet shown in fig. 6-10) and smooth to reduce adhesion between the swelling soil and the shaft. Friction reducing material, such as roofing felt, bitumen slip layers, polyvinyl chloride (PVC), or polyethylene sleeves, may be placed around the upper shaft to re-



(Based on data from U. S. Army Construction Engineering Research Laboratory TR M-81 by W. P. Jobes and W. R. Stroman)

Figure 6-10. Drilled shaft details.

duce both uplift and downdrag forces. Vermiculite, pea gravel, or other pervious materials that will allow access of water to the expansive material should be avoided.

d. Design. The heave or settlement of the foundation usually controls the design and should not exceed specified limits set by usage requirements and tolerances of the structure. The design of drilled shafts, in addition to bearing capacity, should consider the method of construction, skin resistance, and uplift forces. The computer program HEAVE (WES Miscellaneous Paper GL-82-7) may be used to help determine the movement of drilled shafts for different lengths and diameters of the shaft, and the diameter of the underream for different loading forces.

(1) *Skin resistance.* Skin resistance develops from small relative displacements between the shaft and the adjacent soil. Positive (upward directed) skin friction, which helps support structural loads, develops when the shaft moves down relative to the soil. Uplift of adjacent swelling soils also transfers load to the shaft foundation by positive skin friction and can cause large tensile stresses to develop in the shaft. Negative skin friction, which adds to the structural loads and increases the end bearing force, develops when the surrounding soil moves down relative to the shaft, Negative skin friction is associated with the settling of the adjacent fill, loading of surrounding compressible soils, or lowering of the groundwater level.

(*a*) The maximum skin friction f_s may be evaluated by the equation

$$f_s = c_a + K\sigma'_v \tan \phi_a \qquad (6\text{-}2)$$

where

c_a = adhesion, tons per square foot

K = ratio of horizontal to vertical effective stress

σ'_v = vertical effective stress, tons per square foot

ϕ_a = angle of friction between the soil and shaft, degrees

The angle ϕ_a is close to, although less than, the effective angle of internal friction ϕ' for remolded cohesive

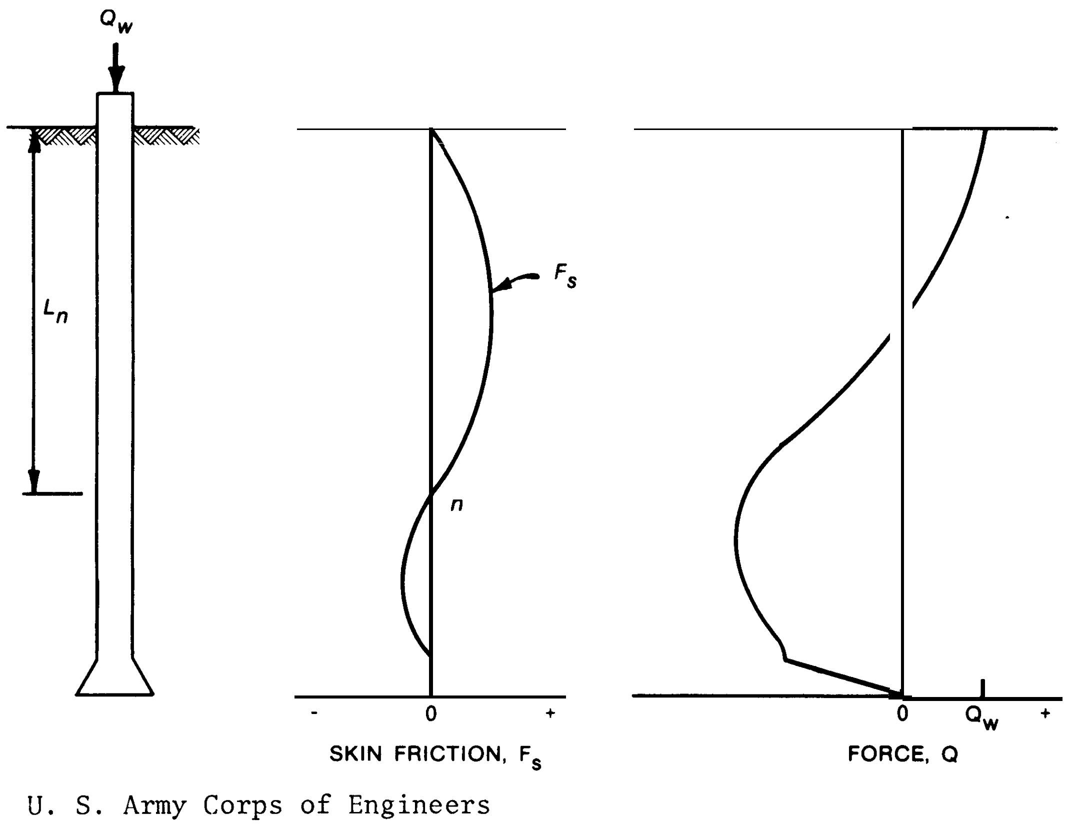


Figure 6-11. Distribution of load from uplift of swelling soil.

soil against concrete. The skin resistance, which is a function of the type of soil (sand, clay, and silt), is usually fully mobilized with a downward displacement of 1/2 inch or less or about 1 to 3 percent of the shaft diameter. These displacements are much less than those required to fully mobilize end bearing resistance.

(b) The fully mobilized skin resistance has been compared with the undrained, undisturbed shear strength c_u for all clays by

$$f_s = c_a = \alpha_f c_u \qquad (6\text{-}3)$$

in which α_f is a reduction coefficient that has been found to vary between 0.25 and 1.25 depending on the type of shaft and soil conditions. The reduction factor is the ratio of mobilized shearing resistance to the undrained, undisturbed shear strength. The α_f appears to be independent of soil strength. Also, the in situ reduction factor may appear greater than one depending on the mechanism of load transfer. For example, the shaft load may be transferred over some thickness of soil such that the effective diameter of the shaft is greater than the shaft diameter D,. The reduction factor concept, although commonly used, is not fully satisfactory because α_f is empirical and should be evaluated for each shaft foundation. The average α_f for use in stiff overconsolidated clays is about 0.5 to 0.6. An α_f of 0.25 is recommended if little is known about the soil or if slurry construction is used.
The reduction factor approaches zero near the top and bottom of the shafts in cohesive soils, reaching a maximum near the center. The reduction of α_f at the top is attributed to soil shrinkage during droughts and low lateral pressure, while the reduction at the bottom is attributed to interaction of stress between end bearing and skin resistance.

(c) Skin resistance may also be evaluated in terms of effective stress from results of drained direct shear tests

$$f_s = c' + K\sigma'_v \tan\phi' = \beta\sigma'_v \qquad (6\text{-}4)$$

where c′ is the effective cohesion and ϕ′ is the effective angle of internal friction. The effective cohesion is assumed zero in practical applications and eliminated from equation (6-4). Most of the available field data show that K tan ϕ′ or β varies from 0.25 to 0.4 for normally consolidated soils, while it is about 0.8 for overconsolidated soils. Reasonable estimates of β can also be calculated for normally consolidated soils by

$$\beta = (1 - \sin\phi')\tan\phi' \qquad (6\text{-}5a)$$

and in overconsolidated soils by

$$\beta = (1 + 2K_o)\frac{\cos\phi'\sin\phi'}{3 - \sin\phi'} \qquad (6\text{-}5b)$$

where K_o is the lateral coefficient of earth pressure at rest. If K_o is not known, a reasonable minimum estimate of β is given by assuming $K_o = 1$. The effective cohesion is often assumed to be zero.

(2) *Uplift forces.* Uplift forces, which area direct function of swell pressures, will develop against sur-

faces of shaft foundations when wetting of surrounding expansive soil occurs. Side friction resulting in uplift forces should be assumed to act along the entire depth of the active zone since wetting of swelling soil causes volumetric expansion and increased pressure against the shaft. As the shaft tends to be pulled upward, tensile stresses and possible fracture of concrete in the shaft are induced, as well as possible upward displacement of the entire shaft.

(a) The tension force T (a negative quantity) may be estimated by

$$T = Q_w - Q_{u1} \tag{6-6}$$

where Q_w is the loading force from the structure and includes the weight of the shaft. Limited observations show that the value of K required to evaluate Q_{u1} (equation (6-1)) from the skin resistance f_s (equations (6-3) and (6-4)) varies between 1 and 2 in cohesive soils for shafts subject to uplift forces. The same swelling responsible for uplift also increases the lateral earth pressure on the shaft. Larger K values increase the computed tension force.

(b) The shaft should be of proper diameter, length, and underreaming, adequately loaded, and contain sufficient reinforcing steel to avoid both tensile fractures and upward displacement of the shaft. ASTM A 615 Grade 60 reinforcing steel with a minimum yield strength f_s of 60,000 psi should be used. The minimum percent steel required if ASTM A 615 Grade 60 steel is used is given approximately by

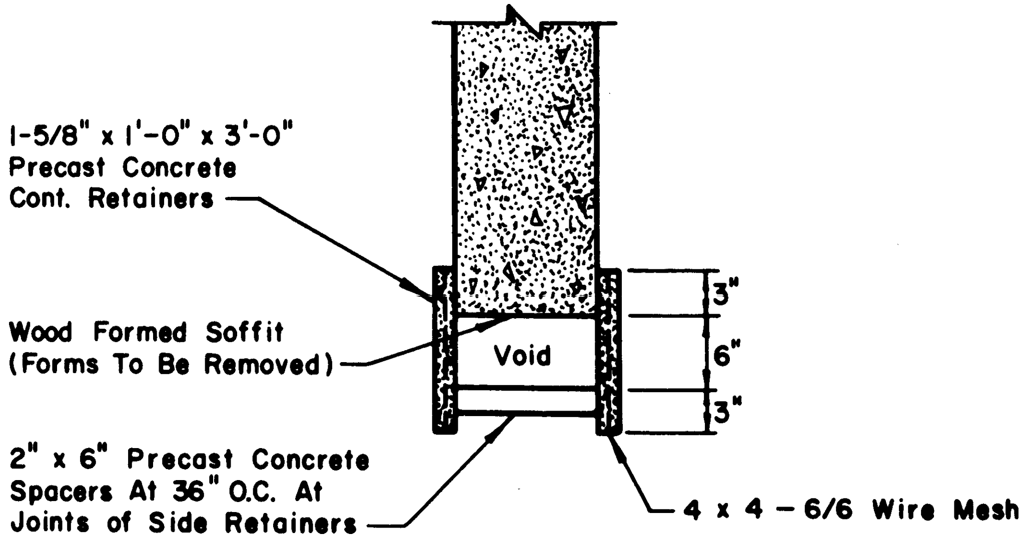
$$\text{Percent } A_s \cong -0.03 \frac{T}{D_s^2} \tag{6-7}$$

where T is the tension force in tons and the shaft diameter D_s is in feet. The minimum percent steel A_s should be 1 percent of the concrete area A_c (fig. 6-10), but more may be required. The reinforcing steel should be hooked into any existing bell as shown in figure 6-10, and it may also be hooked into a concrete grade beam. Maximum concrete aggregate size should be one third of the openings in the reinforcement cage.

d. Grade beams. Grade beams spanning between shafts are designed to support wall loads imposed vertically downward. These grade beams should be isolated from the underlying swelling soil with a void space beneath the beams of 6 to 12 inches or 2 times the predicted total heave of soil located above the base of the shaft foundation (whichever is larger). Steel is recommended in only the bottom of the grade beam if grade beams are supported by drilled shafts above the void space. Grade beams resting on the soil without void spaces are subject to distortion from uplift pressure of swelling foundation soil and are not recommended.

(1) *Preparation of void space.* Construction of grade beams with void spaces beneath the beams may require overexcavation of soil in the bottom of the grade beam trench between shafts. The void space may be constructed by use of sand that must later be blown away at least 7 days after concrete placement, or by use of commercially available cardboard or other retainer forms that will support the concrete. The cardboard forms should deteriorate and collapse before swell pressures in underlying soil can deflect or damage the grade beams. The resulting voids should be protected by soil retainer planks and spacer blocks. Figure 6-12 illustrates some void details.

(2) *Loading.* Interior and exterior walls and concentrated loads should be mounted on grade beams. Floors may be suspended from grade beams at least 6 inches above the ground surface, or they maybe placed directly on the ground if the floor slab is isolated from the walls. Support of grade beams, walls, and suspended floors from supports other than the shaft foundation should be avoided. Figure 6-13 illustrates typical exterior and interior grade beams.



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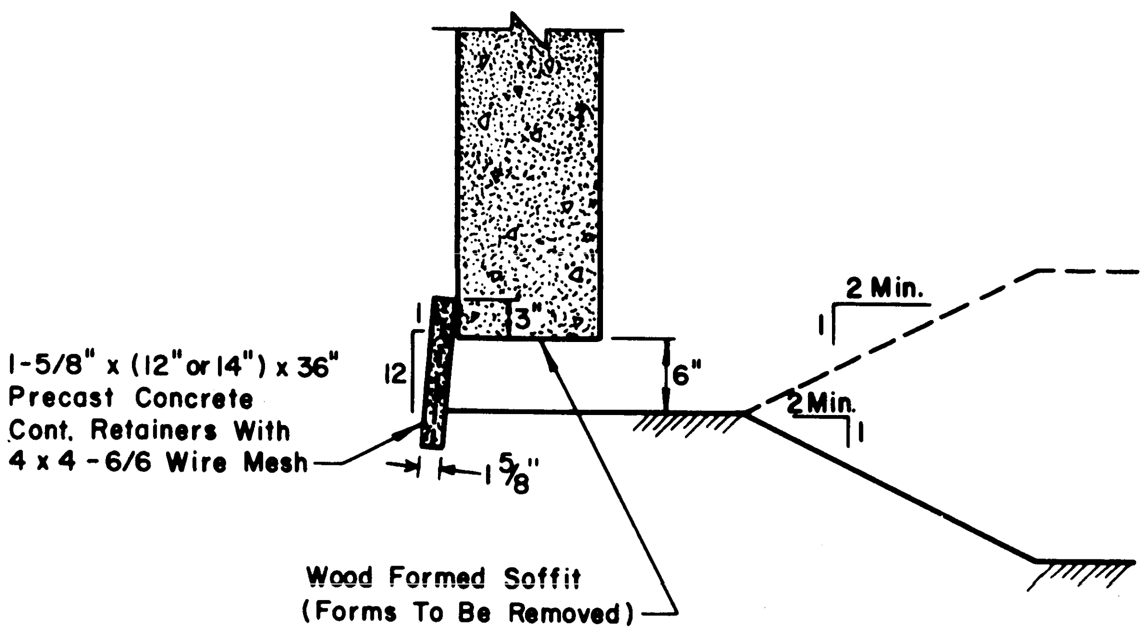

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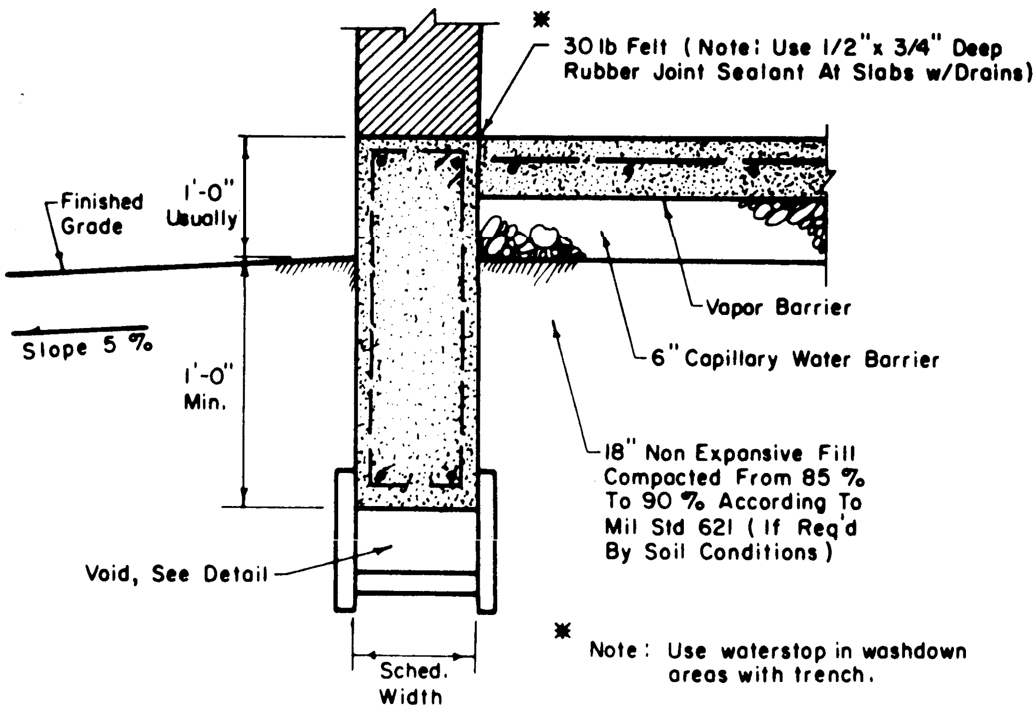
(Based on data from U. S. Army Construction Engineering Research Laboratory TR M-81 by W. P. Jobes and W. R. Stroman)

Figure 6-12. Typical grade beam void details.



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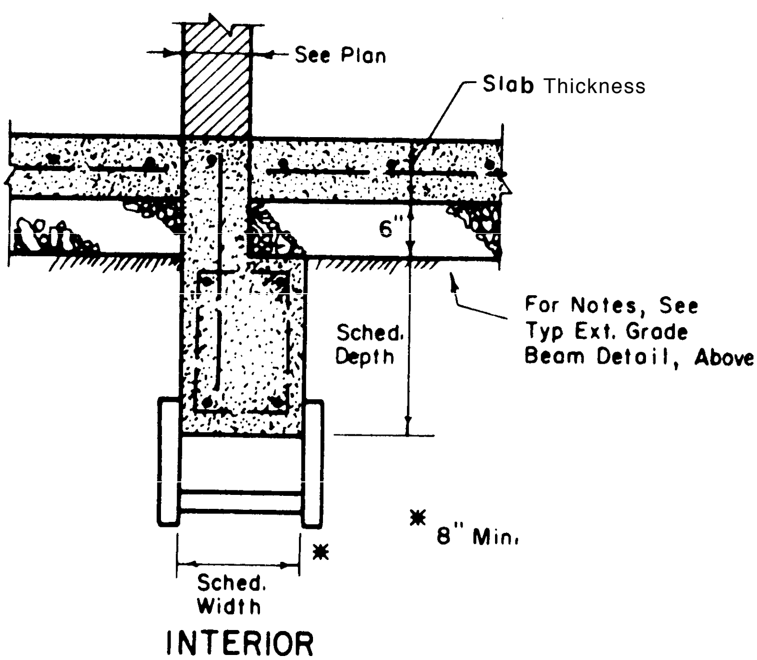

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(Based on data from U. S. Army Construction Engineering Research Laboratory TR M-81 by W. P. Jobes and W. R. Stroman)

Figure 6-13. Typical exterior and interior grade beams.

CHAPTER 7

MINIMIZATION OF FOUNDATION MOVEMENT

7-1. Preparation for construction

The foundation should always be provided with adequate drainage, and the soil properly prepared to minimize changes in soil moisture and differential movement.

a. Removal of vegetation. Existing trees and other heavy vegetation should be removed. New plantings of like items installed during postconstruction landscaping should not be located within a distance away from the structure ranging from 1 to 1.5 times the height of the mature tree.

b. Leveling of site. Natural soil fills compacted at the natural water content and the natural density of the in situ adjacent soil minimize differential movement between cut and fill areas of sloping ground, trenches, or holes caused by removal of vegetation. The volume of cut portions should be kept to a minimum. Cut areas reduce the overburden pressure on underlying swelling soil and lead to time-dependent heave.

c. Excavation.

(1) Construction in new excavations (within a few years of excavating) without replacement of a surcharge pressure equal to the original soil overburden pressure should be avoided where possible because the reduction in effective stress leads to an instantaneous elastic rebound plus a time-dependent heave. The reduction in overburden pressure results in a reduction of the pore water pressure in soil beneath the excavation. These pore pressures tend to increase with time toward the original or equilibrium pore pressure profile consistent with that of the surrounding soil and can cause heave.

(2) Ground surfaces of new excavations, such as for basements and thick mat foundations, should be immediately coated with sprayed asphalt or other sealing compounds to prevent drying of or the seepage of ponded water into the foundation soil during construction (fig. 7-1). Rapid-cure RC 70 or medium-cure MC 30 cutback asphalts are often used as sealing compounds, which penetrate into the soil following compaction of the surface soil and cure relatively quickly.

7-2. Drainage techniques

Drainage is provided by surface grading and subsurface drains.

a. Grading. The most commonly used technique is grading of a positive slope away from the structure. The slope should be adequate to promote rapid runoff and to avoid collecting, near the structure, ponded water, which could migrate down the foundation/soil interface. These slopes should be, greater than 1 percent and preferably 5 percent within 10 feet of the foundation,

(1) Depressions or water catch basin areas should be filled with compacted soil (para 7-3a) to have a positive slope from the structure, or drains should be provided to promote runoff from the water catch basin areas. Six to twelve inches of compacted, impervious, nonswelling soil placed on the site prior to construction of the foundation can ensure the necessary grade and contribute additional uniform surcharge pressure to reduce uneven swelling of underlying expansive soil.

(2) Grading and drainage should be provided for structures constructed on slopes, particularly for slopes greater than 9 percent, to rapidly drain off water from the cut areas and to avoid pending of water in cuts or on the uphill side of the structure. This drainage will also minimize seepage through backfills into adjacent basement walls.

b. Subsurface drains. Subsurface drains (fig. 7-1) may be used to control a rising water table, groundwater and underground streams, and surface water penetrating through pervious or fissured and highly permeable soil. Drains can help control the water table before it rises but may not be successful in lowering the water table in expansive soil. Furthermore, since drains cannot stop the migration of moisture through expansive soil beneath foundations, they will not prevent all of the long-term swelling.

(1) *Location of subsurface drains,* These drains are usually 4- to 6-inch-diameter perforated pipes placed adjacent to and slightly below the baseline of the external wall to catch free water (fig. 7-1).

(a) An impervious membrane should be placed beneath the drain in the trench to prevent migration of surface moisture into deeper soil. The membrane adjacent to the foundation wall should be cemented to the wall with a compatible joint sealant to prevent seepage through the joint between the membrane and the foundation.

(b) If a 6- to 12-inch layer of granular material

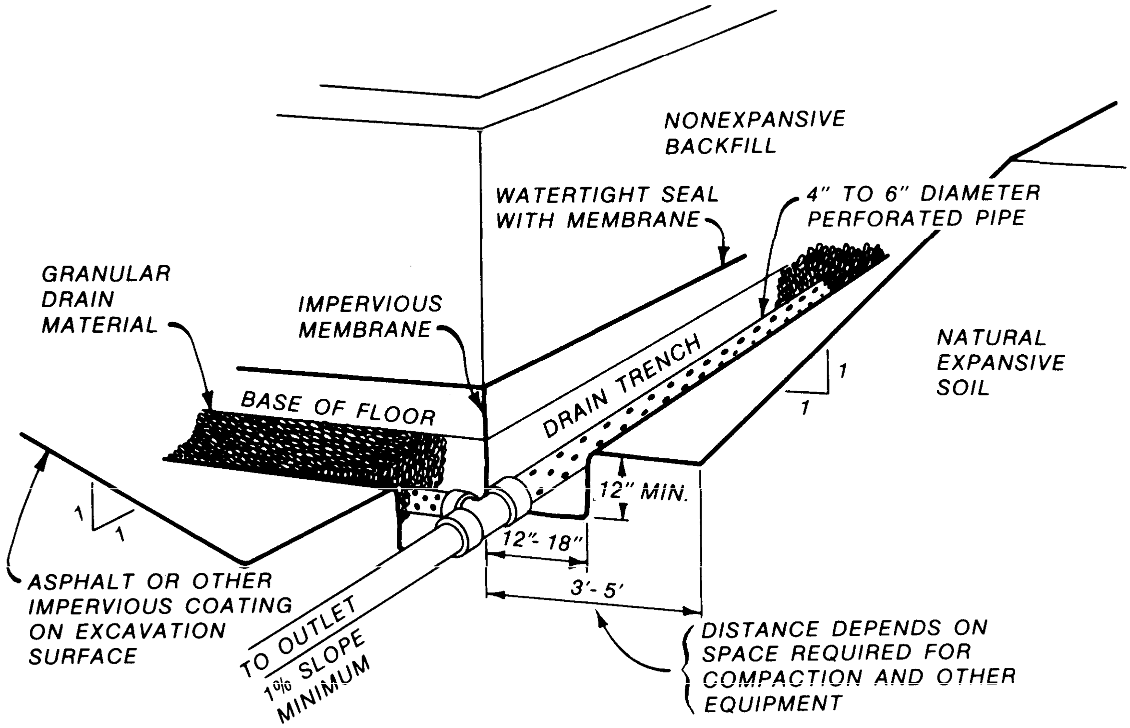

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Figure 7-1. Drainage trench around outside of structure.

was provided beneath a slab-on-grade, the granular material in the drain trench should be continuous with the granular material beneath the floor slab. The perforated pipe should be placed at least 12 inches deeper than the bottom of the granular layer. An impervious membrane should also be placed on the bottom and sides of the drain trench but should not inhibit flow of moisture into the drain from beneath the floor slab. Granular fills of high permeability should be avoided where possible.

(*c*) Deep subsurface drains constructed to control arising water table should be located at least 5 feet below a slab-on-grade. An impervious membrane should not be placed in the drain trench. These drains are only partially effective in controlling soil heave above the drain trench, and they are relatively expensive. A more economical solution may be to place a temporary (or easily removable slab-on-grade) with a permanent slab after the groundwater table has reached equilibrium.

(2) *Outlets.* Drains should be provided with outlets or sumps to collect water and pumps to expel water if gravity drainage away from the foundation is not feasible. Sumps should be located well away from the structure. Drainage should be adequate to prevent any water from remaining in the drain (i.e., a slope of at least 1/8 inch per foot of drain or 1 percent should be provided).

(3) *Drain trench material.* The intrusion of fines in drains maybe minimized by setting the pipe in filter fabric and pea gravel/sand.

7-3. Stabilization techniques

Two effective and most commonly used soil stabilization techniques are controlled backfilling and continuous maintenance involving drainage control and limited watering of surface soil adjacent to the structure during droughts. Other techniques, such as moisture barriers and lime treatment, are not widely used in minimizing differential heave of single and multistory buildings. Presetting or pending for periods of a few months to a year prior to construction is often effective but normally is not used because of time requirements. Prewetting should not be used on fissured clay shales because swelling from water seeping into fissures may not appear until a much later date and delayed problems may result.

a. *Controlled backfills.* Removal of about 4 to 8 feet of surface swelling soil and replacement with nonexpansive, low permeable backfill will reduce heave at the ground surface. Backfills adjacent to foundation walls should also be nonswelling, low permeable material. Nonswelling material minimizes the forces exerted on walls, while low permeable backfill minimizes infiltration of surface water through the backfill into the foundation soil. If only pervious, nonexpansive (granular) backfill is available, a subsurface drain at the bottom of the backfill is necessary to carry off infiltrated water (fig. 7-1) and to minimize seepage of water into deeper desiccated foundation expansive soils.

(1) *Backfill of natural soil.* Backfill using natural soil and compaction control has been satisfactory in some cases if nonswelling backfill is not available. However, this use of backfill should be a last resort,

(a) In general, the natural soil should be compacted to 90 percent of standard maximum density and should be wet of optimum water content. Foundation loads on fills should be consistent with the allowable bearing capacity of the fill. Overcompaction should be avoided to prevent aggravating potentially swelling soil problems such as differential heave of the fill. Compaction control of naturally swelling soil is usually difficult to accomplish in practice. Some soils become more susceptible to expansion following remolding, and addition of water to achieve water contents necessary to control further swell may cause the soil to be too wet to work in the field.

(b) As an alternative, backfills of lime-treated natural soil compacted to 95 percent standard maximum density at optimum water content may be satisfactory if the soil is sufficiently reactive to the lime *(d* below), Lime treatment may also increase soil strength and trafficability on the construction site.

(2) *Backfill adjacent to walls.* A IV on lH slope cut into the natural soil should dissipate lateral swell pressures against basement or retaining walls exerted by the natural swelling material. The nonswelling backfill should be a weak material (sand fill with friction angle of 30 degrees or lessor cohesive fill with cohesion less than about 0.5 tons per square foot) to allow the fill to move upward when the expansive natural soil swells laterally. Restraining loads should not be placed on the surface of the fill. A friction reducing medium may be applied on the wall to minimize friction between the wall and the backfill, TM 5-818-4 discusses details on optimum slopes of the excavation and other design criteria.

b. Maintenance. Maintenance programs are directed toward promoting uniform soil moisture beneath the foundation. A good program consists of the following:

(1) Maintenance of a positive slope of about 5 percent around the structure for drainage and elimination of water catch areas.

(2) Maintenance of original drainage channels and installation of new channels as necessary.

(3) Maintenance of gutters around the roof and diversion of runoff away from the structure.

(4) Avoidance of curbs or other water traps around flower beds.

(5) Elimination of heavy vegetation within 10 to 15 feet of the foundation or 1 to 1.5 times the height of mature trees.

(6) Uniform limited watering around the structure during droughts to replace lost moisture.

c. Moisture barriers. The purpose of moisture barriers is to promote uniform soil moisture beneath the foundation by minimizing the loss or gain of moisture through the membrane and thus reducing cyclic edge movement, Moisture may still increase beneath or within areas surrounded by the moisture barriers leading to a steady but uniform heave of the foundation or slab-on-grade.

(1) *Types of barriers.* These barriers consist of horizontal and vertical plastic and asphalt membranes and granular materials. Concrete is an ineffective moisture barrier. Longlasting membranes include chlorinated polyethylene sheets, preferably placed over a layer of catalytically blown or sprayed asphalt. All joints, seams, and punctures should be sealed by plastic cements or concrete/asphalt joint sealants. ASTM D 2521 (Part 15) describes use of asphalt in canal, ditch, and pond linings (app A).

(2) *Horizontal.*

(a) An impervious membrane on the ground surface in a crawl space where rainfall does not enter may help reduce shrinkage in clayey foundation soils with deep groundwater levels by minimizing evaporation from the soil. A vapor barrier should not be placed in ventilated crawl spaces if there is a shallow water table or if site drainage is poor because heave maybe aggravated in these cases. Figure 7-2 illustrates a useful application of horizontal membranes,

(b) Other applications include the use of horizontal moisture barriers around the perimeter of structures to reduce lateral variations in moisture changes and differential heave in the foundation soil. Plastic or other thin membranes around the perimeter should be protected from the environment by a 6- to 12-inch-thick layer of earth.

(c) A disadvantage of these barriers is that they are not necessarily reliable and may be detrimental in some cases. For example, most fabrics and plastic membranes tend to deteriorate with time. Undetected (and hence unrepaired) punctures that allow water to get in, but not to get out, commonly occur in handling on placement. Punctures may also occur during planting of vegetation. If the barrier is a concrete slab, the concrete may act as a wick and pull water out of the soil.

(3) *Vertical.*

(a) Plumbing or utility trenches passing through the barrier may contribute to soil moisture beneath the foundation.

(b) The vertical barrier (fig. 7-3) should extend to the depth of the active zone and should be placed a minimum of 3 feet from the foundation to simplify construction and to avoid disturbance of the foundation soil. The barrier may not be practical in prevent-

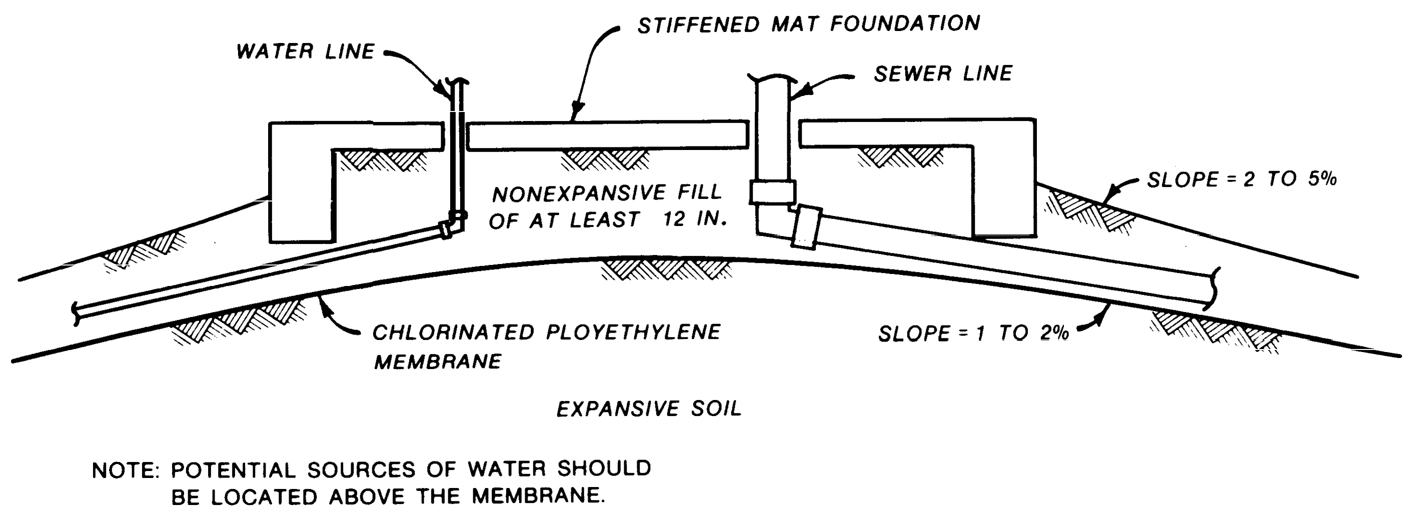

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Figure 7-2. Application of a horizontal membrane.

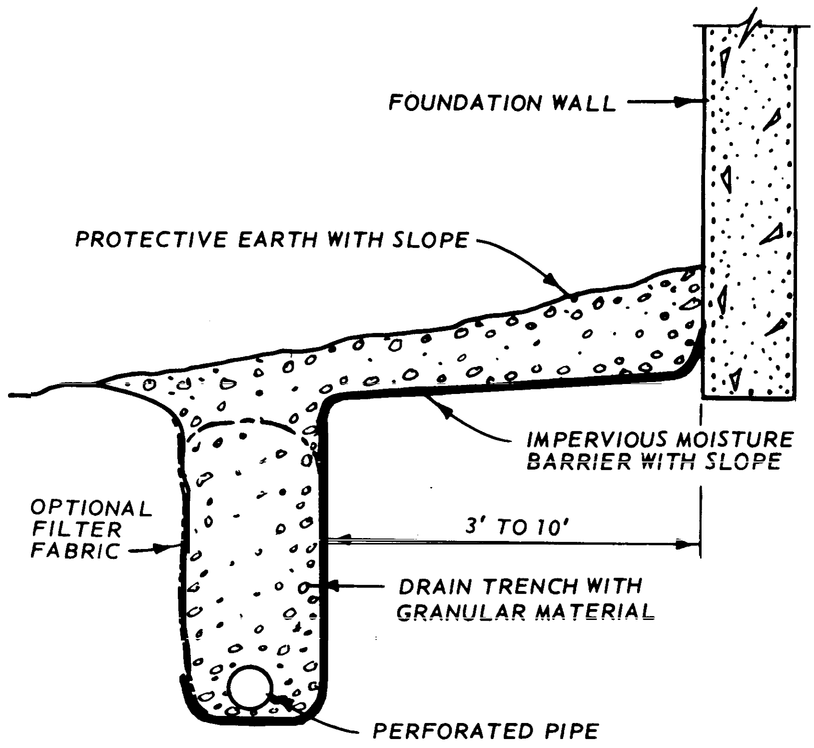

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Figure 7-3. Vertical and horizontal moisture barriers.

ing migration of moisture beneath the bottom edge for active zones deeper than 8 to 10 feet. The granular barrier may also help reduce moisture changes during droughts by providing a reservoir of moisture. The placement of a filter fabric around the trench to keep fine particles from entering the perforated pipe will permit use of an open coarse aggregate instead of a graded granular filter. In some cases, the perforated pipe could be eliminated from the drain trench.

d. Lime treatment. This treatment is the most widely used and most effective technique of chemical alteration to minimize volume changes and to increase the shear strength of foundation expansive soils.

(1) *Applications.* Lime treatment is applied to the strengthening and minimization of volume change of soil in railroad beds, pavement subgrades, and slopes. When this treatment is applied to foundation soils of single and multistory structures, it is not always successful because the usefulness depends on the reactiveness of the soil to lime treatment and the thoroughness of dispersion of lime mixed into the soil.

(a) Lime treatment is effective in the minimization of volume changes of natural soil for backfill. However, this treatment increases the soil permeability and the soil strength. The soil permeability should be kept low to restrict seepage of surface water through the backfill. The backfill strength should be as low as possible compatible with economical design to minimize the transfer of lateral swell pressures from the natural in situ soil through the backfill to the basement and retaining walls.

(b) Lime treatment may be used to stabilize a 6- to 12-inch layer of natural expansive soil compacted on the surface of the construction site to provide a positive slope for runoff of water from the structure and a layer to reduce differential heave beneath the floor slab.

(c) Lime treatment may be applied to minimize downhill soil creep of slopes greater than 5 degrees (9 percent) by increasing the stiffness and strength of the soil mass through filling fractures in the surface soils. If lime slurry pressure injection (LPSI) can cause a lime slurry to penetrate the fissures in the soil mass to a sufficient depth (usually 8 to 10 feet), then the lime-filled seams will help control the soil water content, reduce volumetric changes, and increase the soil strength. However, LSPI will probably not be satisfactory in an expansive clay soil that does not contain an extensive network of fissures because the lime will not penetrate into the relatively impervious soil to any appreciable distance from the injection hole to form a continuous lime seam moisture barrier.

(d) LSPI may be useful for minimization of movement of fissured foundation expansive soils down to the depth of the active zone for heave or at least 10 ft. The lime slurry is pressure injected on 3- to 5-foot center to depths of 10 to 16 feet around the perimeter of the structure 3 to 5 feet from the structure.

(2) *Soil mixture preparation.* Lime should be thoroughly and intimately mixed into the soil to a sufficient depth to be effective. For stabilization of expansive clay soils for foundations of structures, mixing should be done down to depths of active zone for heave. In practice, mixing with lime is rarely done deeper than 1 to 2 feet. Therefore, lime treatment is normally not useful for foundations on expansive soil except in the above applications. Moreover, poor mixing may cause the soil to break up into clods from normal exposure to the seasonal wetting/drying cycles. The overall soil permeability is increased and provides paths for moisture flow that require rapid drainage from this soil. Lime treatment should be performed by experienced personnel.

(3) *Lime modification optimum content (LMO).* The LMO corresponds to the percent of lime that maximizes the reduction in the soil plasticity or the PI. The reduction in plasticity also effectively minimizes the volume change behavior from changes in water content and increases the soil shear strength.

(a) A decision to use lime should depend on the degree of soil stabilization caused by the lime. Lime treatment is recommended if a 50 percent reduction in the PI is obtained at the LMO content (table 7- 1). The PI should be determined for the natural soil, LMO, LMO+ 2, and LMO - 2 percent content.

(b) The increase in strength of the lime-treated soil should be similar for soil allowed to cure at least 2 or more days following mixing and prior to compaction to similar densities.

(c) The amount of lime needed to cause the optimum reduction in the PI usually varies from 2 to 8 percent of the dry soil weight.

e. Cement treatment. Cement may be added to the soil to minimize volume changes and to increase the shear strength of the foundation expansive soil if the degree of soil stabilization achieved by lime alone is not sufficient. The amount of cement required will probably range between 10 to 20 percent of the dry soil weight. A combination of lime-cement or lime-cement-fly ash may be the best overall additive, but the best combination can only be determined by a laboratory study. TM 5-822-4 presents details on soil stabilization with cement and cement-lime combinations.

Table 7-1. pH Test Procedure for the Lime Modification Optimum Content

Materials:

1. Lime to be used for soil stabilization.
2. Air-dried soil.

Apparatus:

1. pH meter (the pH meter must be equipped with an electrode having a pH range of 14).
2. 150-ml (or larger) plastic bottles with screw-top lids.
3. 50-ml plastic beakers.
4. CO_2—free distilled water.
5. Balance.
6. Oven.
7. Moisture cans.

Procedure:

1. Standardize the pH meter with a buffer solution having a pH of 12.45.
2. Weigh to the nearest 0.01 g representative samples of air-dried soil, passing the No. 40 sieve and equal to 20.0 g of oven-dried soil.
3. Pour the soil samples into 150-ml plastic bottles with screw-top lids.
4. Add varying percentages of lime, weighed to the nearest 0.01 g, to the soils. (Lime percentages of 0, 1, 2, 3, 4, 5 6, and 8, based on the dry soil weight, may be used.)
5. Thoroughly mix soil and dry lime.
6. Add 100 ml of CO_2—free distilled water to the soil-lime mixtures.
7. Shake the soil-lime and water for a minimum of 30 sec or until there is no evidence of dry material on the bottom of the bottle.
8. Shake the bottles for 30 sec every 10 min.
9. After 1 hr, transfer parts of the slurry to a plastic beaker and measure the pH.
10. Record the pH for each of the soil-lime mixtures. The lowest percent of lime giving a pH of 12.40 is the percent required to stabilize the soil. If the pH does not reach 12.40, the minimum lime content giving the highest pH is that required to stabilize the soil.

CHAPTER 8

CONSTRUCTION TECHNIQUES AND INSPECTION

8-1. Minimization of foundation problems from construction

Many problems and substandard performance of foundations observed in structures on expansive soils occur from poor quality control and faulty construction practice. Much of the construction equipment and procedures that are used depends on the foundation soil characteristics and soil profiles. Careful inspection during construction is necessary to ensure that the structure is built according to the specifications.

a. *Important elements of construction techniques.* Construction techniques should be used that promote a constant moisture regime in the foundation soils during and following construction. The following elements of construction are important in obtaining adequate foundation performance in expansive soils.

(1) *Excavations.* The excavation should be completed as quickly as possible to the design depth and protected from drying. An impervious moisture barrier should be applied on the newly exposed surfaces of the excavation to prevent drying of the foundation soils immediately after excavating to the design depth. Sides of the excavation should be constructed on a lV on lH slope or an appropriate angle that will not transmit intolerable swelling pressures from the expansive soil to the foundation. The foundation should be constructed in the excavation as quickly as practical.

(2) *Selection of materials.* Selected materials should conform to design requirements.

(a) Backfills should be nonswelling materials.

(b) Concrete should be of adequate strength and workability.

(c) Reinforcing steel should be of adequate size and strength.

(d) Moisture barriers should be durable and impervious.

(3) *Placement of materials.* All structural materials should be positioned in the proper location of the foundation.

(4) *Compaction of backfills.* Backfills of natural expansive soil should be compacted to minimize effects of volume changes in the fill on performance of the foundation. Backfills should not transmit intolerable swell pressures from the natural expansive foundation soil to basement or retaining walls.

(5) *Drainage during construction.* The site should be prepared to avoid ponding of water in low areas. Consideration should be given to compaction of 6 to 12 inches or more of impervious nonswelling soil on the site prior to construction of the foundation to promote drainage and trafficability on the site. Dehydrated lime may also be sprinkled on the surface of expansive soil to promote trafficability. Sumps and pumps should be provided at the bottom of excavations if necessary to remove rainwater or subsurface drainage entering the excavation. Provision for after normal duty operation of the pumps should also be made.

(6) *Permanent drainage.* Grades of at least 1 percent and preferably 5 percent, to promote drainage of water away from the structure, should be provided around the perimeter of the structure. Low areas should be filled with compacted backfill. Runoff from roofs should be directed away from the structure by surface channels or drains. Subsurface drains should be constructed to collect seepage of water through pervious backfills placed adjacent to the foundation.

b. *Considerations of construction inspection.* Table 8-1 lists major considerations of construction inspection. Inspections related to concrete reinforced slab and drilled shaft foundations, the two most commonly used foundations in expansive soil areas, are discussed below.

8-2. Stiffened slab foundations

Items in table 8-2 should be checked to minimize defective slab foundations.

a. The inspector should check for proper site prep aration and placement of the moisture barrier, steel, and concrete. All drainage systems should be inspected for proper grade and connections to an outlet.

b. Posttensioned slabs require trained personnel and careful inspection to properly apply the posttensioning procedure. For example, anchors for the steel tendons should be placed at the specified depth (lower than the depth of the tensioning rods) to avoid pullout during tensioning. Tendons should be stressed 3 to 18 days following the concrete placement (to eliminate much of the shrinkage cracking) such that the minimum compressive stress in the concrete exceeds 50 pounds per square inch. Stressing should be completed before structural loads are applied to the slabs.

8-3. Drilled shaft foundations

Items in table 8-3 should be checked to minimize defective shaft foundations. The foundation engineer should visit the construction site during boring of the first shaft holes to verify the assumptions regarding the subsurface soil profile, e.g., the nature and location of the subsoils. Periodically, he or she should also check the need for the designer to consider modifications in the design.

a. Location of shaft base. The base of the shaft is located in the foundation soils to maintain shaft movements within tolerable limits. This depth depends on the location and thickness of the expansive, compressible or other unstable soil, sand lenses or thin permeable zones, depth to groundwater, and depth to foundation soil of adequate bearing capacity. The design depth may require modification to relocate the base in the proper soil formation of adequate bearing capacity and below the active zone of heave. The purpose of locating the base of the shaft in the proper soil formation should be emphasized to the inspector during the first boring of the drilled shaft foundation. Underreams may be bored in at least 1.5-foot-diameter (preferably 2.5-foot) dry or cases holes where inspections are possible to ensure cleanliness of the bottom.

b. Minimization of problems, Long experience has shown that drilled shaft foundations are reliable and economical. Nevertheless, many problems are associated with these foundations and can occur from inadequate understanding of the actual soil profile and groundwater conditions, mistakes made while drilling, inadequate flow of concrete, and improper reinforcement.

(1) *Inadequate information.*

(a) Site conditions should be known to permit optimum selection of equipment with the required mobility.

(b) Subsurface conditions should be known to permit selection of equipment with adequate boring capacity.

(c) Type of soil (e.g., caving and pervious strata) may require slurry drilling. Specifications should permit sufficient flexibility to use slurry for those soil conditions where it maybe needed.

(d) Previously unnoticed sand lenses or thin permeable zones in otherwise impervious clay may cause problems during construction of drilled shafts. Seepage through permeable zones may require casing or slurry and may render construction of an underream nearly impossible.

(e) Overbreak or the loss of material outside of the nominal diameter of the shaft due to caving soil is a serious problem that can cause local cavities or defects in the shaft. The construction procedure (boring dry, with casing, or using slurry) should be chosen to minimize overbreak.

(2) *Problems with the dry method.* Caving, squeezing soil, and seepage are the most common problems of this method. Stiff or very stiff cohesive soils with no joints or slickensides are usually needed. Underreams are vulnerable to caving and should be constructed as quickly as possible.

(3) *Problems with the casing method.* Slurry should be used while drilling through caving soil prior to placement of the casing and sealing of the casing in an impervious layer. An impervious layer is necessary to install the bottom end of the casing.

(a) Casing should not be pulled until the head of concrete is sufficient to balance the water head external to the casing; otherwise, groundwater may mix with the concrete.

(b) Squeezing or localized reduction in the borehole diameter on removal of the casing can be minimized by using a relatively high slump concrete with a sufficient pressure head.

(c) Casing sometimes tends to stick in place during concrete placement. If the concrete appears to be setting up, attempts to shake the casing loose should be abandoned and the casing left in place to avoid the formation of voids in the shaft when the casing is pulled.

(d) Steel reinforcement should be full length to avoid problems in downdrag of the reinforcement while the casing is pulled. The reinforcement cage should also be full length if uplift forces are expected on the drilled shaft from swelling soil.

(4) *Problems with the slurry method.* Slurry of sufficient viscosity is used to avoid problems with caving soils. A rough guide to appropriate slurry viscosities is given by a Marsh cone funnel test time of about 30 seconds for sandy silts and sandy clays to 50 seconds for sands and gravels. The Marsh cone test time is the time in seconds required to pour 1 quart of slurry through the funnel. The workability of the slurry should also be adequate to allow complete displacement of the slurry by the concrete from the perimeter of the borehole and steel of the rebar cage.

(a) Slurries should be of sufficient viscosity to eliminate settling of cuttings. Loose cuttings adhering to the perimeter of the hole can cause inclusions and a defective shaft.

(b) The tremie sometimes becomes plugged, stopping the flow of concrete into the borehole. The tremie should not be pulled above the concrete level in the shaft before the concrete placement is completed, otherwise inclusions may occur in the shaft following reinsertion of the tremie into the concrete.

(c) The reinforcement cage may move up if the tremie is too deep in the concrete or the concrete is placed too rapidly.

c. Placement of concrete. Concrete strength of at least 3,000 pounds per square inch should be used and placed as soon as possible on the same day as drilling the hole. Concrete slumps of 4 to 6 inches and limited

aggregate size of one third of the rebar spacing are recommended to facilitate flow of concrete through the reinforcement cage and to eliminate cavities in the shaft. Care should be exercised while placing the concrete to ensure the following:

(1) Continuity while pulling the casing.

(2) Tip of tremie always below the column of freshly placed concrete in wet construction; no segregation in a dry hole.

(3) Adequate strength of the rebar cage to minimize distortion and buckling.

Table 8-1. Considerations for Inspection

	Construction
Excavation	1. Bracing system. 2. Tie backs. 3. Dewatering. 4. Retaining structures. 5. Protection from drying of temporarily exposed surfaces of expansive clay.
Effects on surrounding structures	1. Retaining walls and lost ground. 2. Slope stability, erosion, and soil stabilization. 3. Surface and subsurface drainage. 4. Foundation movement and fracture of adjacent (nearby older structures).
Maintenance	1. Broken or leaking water, sewer, and other utility lines. 2. Surface drainage system. 3. Vibration effects from adjacent (nearby) structures. 4. Changes in groundwater.
	Postconstruction
	1. Broken or leaking water, sewer, and other utility lines. 2. Surface drainage system. 3. Foundation movement and fractures in the new structure. 4. Vibration effects from adjacent (nearby) structures. 5. Changes in groundwater. 6. Heavy vegetation near the structure.

Table 8-2. Inspection of Reinforced Slab Foundations

Site preparation	1. Proper selection of materials. 2. Proper compaction of fill. 3. Proper backfill of plumbing trenches and holes due to removal of trees. 4. Proper cleanout of trenches for reinforcing beams. 5. Proper slope of trenches. 6. New excavations coated with sprayed asphalt or sealing surface to prevent drying of the exposed excavation surface. 7. Proper beam size and spacing. 8. Proper slab thickness.
Membrane placement	1. Moisture barrier contoured to the shape of the trench to eliminate voids between the trench and bottom of the membrane. 2. Elimination of punctures, holes, and leaks in the membrane.
Steel placement	1. Proper location of steel reinforcing bars and wire mesh. 2. Proper placement of tensioning rods and anchors. 3. Proper reinforcement size. 4. Adequate forming and means to hold post-tensioning anchorage assemblies in place.
Concrete placement	1. Mixture as specified (e.g., approved components in mixture, desired slump of concrete, no extra water added to mixture, proper conveying, placing and vibrating of concrete, and finishing). 2. Reinforcement not displaced by concrete. 3. Provide adequate curing for slab. 4. Obtain desired early age strength of concrete before form removal and before allowing traffic on the slab.

Table 8-2. Inspection of Reinforced Slab Foundations—Continued

Post-tensioning	1. Verify all tendons stressed according to specification and within 3 to 10 days of the concrete placement. 2. Ends of properly stressed tendons cut off, pockets grouted, and any necessary repairs made. Improperly stressed tendons must not be cut off.

Table 8-3. Inspection of Drilled Shafts

Drilling	1. Proper shaft dimensions. 2. Collapse of hole. 3. Proper cleanout of hole of loose cuttings and weak soil.
Dry method	1. Loose cuttings in the hole. 2. No more than 2 to 3 inches of water at the bottom if end bearing. 3. Concrete not strike the shaft perimeter if free fall (ACI 304-73 recommends that concrete should be deposited at or near its final position such that the tendency to segregate is eliminated when flowing laterally into place). 4. Adequate vibration provided to consolidate concrete around reinforcement.
Casing method	1. Clean and undeformed casing before concrete placement. 2. Sufficient concrete placed to balance the external pressure head before the casing is pulled.
Slurry method	1. Viscosity of slurry adequate to be displaced from the perimeter of the hole and the reinforcing steel by the concrete. 2. Clean-out bucket used to clean the bottom prior to concreting. 3. Bottom of the tremie pipe maintained in fresh concrete at all times after placement has begun. The deeper the embedment in concrete the flatter the finished slope will be.
Underreams	1. Minimal cuttings in the bottom (at least 75 to 80 percent of the bottom free of cuttings). 2. Adequate bell diameter (check travel of the kelly on the ground surface when the reamer is extended to the proper bell diameter).
Concrete placement	1. No segregation during placement. 2. Concrete never to be poured through water. 3. Adequate slump (avoid hot concrete). 4. Maximum aggregate size not too large for reinforcement.
Reinforcement cage	1. Resistance to buckling during the concrete placement. 2. Full length if casing used. 3. Restriction to flow of concrete through the cage. 4. Restrained from movement during concrete placement. 5. Proper position of the cage.

CHAPTER 9

REMEDIAL PROCEDURES

9-1. Basic considerations

Remedial work for damaged structures is usually difficult to determine because the cause of the problem (e.g., location of source or loss of soil moisture, and swelling or settling/shrinking soil) may not be readily apparent, A plan to fix the problem is often difficult to execute, and the work may have to be repeated because of failure to isolate the cause of the moisture changes in the foundation soil, An effective remedial procedure may not be found until several attempts have been made to eliminate the differential movement. Requirements for minimizing moisture changes (chap. 7) are therefore essential. The foundation should have sufficient capacity to maintain all distortion within tolerable limits acceptable to the superstructure. This distortion occurs from differential heave for the most severe climates and changes in the field environment.

a. Specialized effort. Investigation and repair are therefore specialized procedures that usually require much expertise and experience. Cost of repair work can easily exceed the original cost of the foundation. The amount of damage that requires repair also depends on the attitudes of the owner and occupants to tolerate distortion as well as damage that actually impairs the usefulness and safety of the structure.

b. Minimization of repairs. Most damage from effects of swelling soil tends to be cosmetic rather than structural, and repairs are usually more economical than rebuilding as long as the structure remains sound. At-early signs of distress, remedial action to minimize future distortion should be undertaken and should be given a greater priority than the cosmetic repairs as this action will minimize maintenance work over the long term. Maintenance expenses and frequency of repairs tend to be greatest in lightly loaded structures and residences about 3 to 4 years following the original construction. Overall maintenance can be minimized by taking remedial action to minimize future distortion before extensive repairs are required (e.g., breaking out and replacing sections of walls).

c. *Examples of remedial procedures.* The choice of remedial measures is influenced by the results of site and soil investigations as well as by the type of original construction. Table 9-1 illustrates common remedial measures that can be taken. Only one remedial procedure should be attempted at a time so as to determine its effect on the structure. The structure should be allowed to adjust, following completion of remedial measures, for at least a year before cosmetic work is done. The structure is seldom rebuilt to its original condition, and in some instances, remedial measures have not been successful.

9-2. Evaluation of information

All existing information on the foundation soils and design of the foundation and superstructure should be studied before proceeding with new soil investigations.

a. Foundation conditions. The initial soil moisture at time of construction, types of soil, soil swell potentials, depth to groundwater, type of foundation and superstructure, and drainage system should be determined. The current soil moisture profile should also be determined. Details of the foundation, such as actual bearing pressures, size and length of footings, and slab and shaft reinforcing, should also be collected. Drilling logs made during construction of shaft foundations may be used to establish soil and groundwater conditions and details of shaft foundations. Actual construction should be checked against the plans to identify any variances.

b. Damages. The types and locations of damage, as well as the time movements first became noticeable, should be determined, Most cracks caused by differential heave are wider at the top than at the bottom. Nearly all lateral separation results from differential heave. Diagonal cracks can indicate footing or drilled shaft movement, or lateral thrust from the doming pattern of heaving concrete slabs. Fractures in slabs-on-grade a few feet from and parallel with the perimeter walls also indicate heaving of underlying soils. Level surveys can be used to determine the trend of movement when prior survey records and reliable benchmarks are available. Excavations may be necessary to study damage to deep foundations, such as cracks in shafts from uplift forces.

c. Sources of moisture. The source of soil moisture that led to the differential heave should be determined to evaluate the cause of damage. Location of deeprooted vegetation, such as shrubs and trees, location and frequency of watering, inadequate slopes and pending, seepage into foundation soil from surface or perched water, and defects in drain, water, and sewer lines can make important changes in soil moisture and can lead to differential heave.

9-3. Stiffened slab foundations

Most slab foundations that experience some distress are not damaged sufficiently to warrant repairs. Damage is often localized by settlement or heave of one side of the slab. The cause of the soil movement, whether settlement or heave, should first be determined and then corrected.

a. Stabilization of soil moisture. Drainage improvements and a program to control soil moisture at the perimeter of the slab are recommended (chap 7) for all damaged slab foundations.

b. Remedial procedures. Remedial work on slabs depends on the type of movement, Repair of a settled area requires raising of that area, while repair of a heaved area often requires raising the entire unheaved portion of the slab up to the level of the heaved portion. Repair costs are consequently usually greater for heaving than settling cases.

(1) Repair of a damaged slab consists of a combination of underpinning and mudjacking using a cement grout. Mudjacking using a cement grout is required simultaneously with underpinning to fill voids during leveling of the slab. Fractured slabs are usually easier to repair than unfractured slabs that have been distorted by differential movement because usually only the fractured portion of the slab requires treatment. The distortion of unfractured slabs can also cause considerable damage to the superstructure and inconvenience to the occupants.

(2) Underpinning and mudjacking are applied simultaneously and usually clockwise around the slab until all parts of the foundation are at the same elevation. If a heaved area is lowered to the same elevation as the rest of the foundation, such as to repair a mushroomed or dome-shaped heave pattern, the slab is first supported before digging out the soil to prevent the slab from creeping down on the work crew during the digging. Attempts at leveling dome-shaped distortion by raising the perimeter may be unsuccessful because mudjacking usually causes the entire slab to rise.

9-4. Drilled shaft foundations

Most damage to structures with shaft foundations consists of fractured slabs-on-grade. The shaft may contribute to the damage caused by migration of moisture down the shaft/soil interface into swelling soil beneath the shaft footing. The fracture pattern of open cracks in the floor slab parallel to and several feet from the wall often shows that the slab had not been free to move near the walls. Damage to drilled shafts is often caused by upward movement of the shaft from swelling soil beneath its base and by uplift forces on the shaft perimeter from adjacent swelling soil.

a. Stabilization of soil moisture. Drainage improvements and a program to control soil moisture around the perimeter of the foundation are recommended (chap 7).

b. Remedial procedures. Repair often requires total removal of the slab and underlying wet soil, replacement with nonswelling soil, and placement of a new slab isolated from the perimeter walls. Repair of drilled shafts consists of cutting down the top of the shaft and releveling the foundation. The tops of the drilled shafts are cut to the elevation of the top of the lowest shaft where possible.

Table 9-1. Remedial Measures

Measure	Description
Drainage	Slope ground surface (positive drainage) from structure; add drains for downspouts and outdoor faucets in areas of poor drainage, and discharge away from foundation soil; provide subdrains if perched water tables or free flow of subsurface water are problems; provide flexible, watertight utility connections.
Moisture stabilization (maintenance of constant moisture whether at high or low levels)	Remove natural swelling soil and recompact with impervious, nonswelling backfill; install vertical and/or horizontal membranes around the perimeter; locate deep-rooted vegetation outside of moisture barriers; avoid automatic sprinkling systems in areas protected with moisture barriers; provide a constant source of moisture if a combination of swelling/shrinking soils is occurring; thoroughly mix 4 to 8 percent lime into soil to reduce potential for swell or pressure-inject line slurry around the perimeter of the structure.
Superstructure adjustments	Free slabs from foundation by cutting along foundation walls; provide slip joints in interior walls and door frames; reinforce masonry and concrete block walls with horizontal and vertical tie bars or reinforced concrete beams; provide fanlights over doors extended to the ceiling.

Table 9-1. Remedial Measures–Continued

Measure	Description
Spread footings and deep foundation adjustments	Decrease footing size; underpin with deep shafts; mudjack using a cement grout; reconstruct void beneath grade beams; eliminate mushrooms at top of shafts; adjust elevation by cutting the top of the shaft or by adding shims; increase footing or shaft spacing to concentrate loading forces and to reduce angular distortion from differential heave between adjacent footings and shafts.
Continuous wall foundation adjustments	Provide voids beneath portions of wall foundation; posttension; reinforce with horizontal and vertical tie bars or reinforced concrete beams.
Reinforced and stiffened slab-on-grade adjustments	Mudjack using a cement grout; underpin with spread footings or shafts to jack up the edge of slabs,

APPENDIX A

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APPENDIX B

CHARACTERIZATION OF SWELL BEHAVIOR FROM SOIL SUCTION

B-1. Introduction

Soil suction is a quantity that can be used to characterize the effect of moisture on volume, and it is a measure of the energy or stress that holds the soil water in the pores or a measure of the pulling stress exerted on the pore water by the soil mass. The total soil suction is expressed as a positive quantity and is defined as the sum of matrix τ_m° and osmotic τ_s suctions.

a. Matrix suction. The matrix suction τ_m° is related to the geometrical configuration of the soil and structure, capillary tension in the pore water, and water sorption forces of the clay particles. This suction is also pressure-dependent and assumed to be related to the in situ pore water pressure u_w by

$$\tau_m^\circ = -u_w + \alpha \delta_m \qquad \text{(B-1)}$$

$$\delta_m = \frac{1 + {}^{2K}T}{3} \delta_v \qquad \text{(B-2)}$$

where

τ_m° = matrix soil suction, tons per square foot
α = compressibility factor, dimensionless
δ_m = total mean normal confining pressure, tons per square foot
K_T = ratio of total horizontal to vertical stress in situ
δ_v = total vertical pressure, tons per square foot

The exponent "°" means that the τ_m° is measured without confining pressure except atmospheric pressure. Experimental results show that the in situ matrix suction τ_m is equivalent to $-u_w$ for soils. The compressibility factor is determined by the procedure in paragraph B-3d.

b. Osmotic suction. The osmotic suction τ_s is caused by the concentration of soluble salts in the pore water, and it is pressure-independent. The effect of the osmotic suction on swell is not well known, but an osmotic effect may be observed if the concentration of soluble salts in the pore water differs from that of the externally available water. For example, swell may occur in the specimen if the external water contains less soluble salts than the pore water. The effect of the osmotic suction on swell behavior is assumed small compared with the effect of the matrix suction. The osmotic suction should not significantly affect heave if the salt concentration is not altered.

B-2. Methods of measurement

Two methods are recommended for determining the total soil suction: thermocouple psychrometer and filter paper. The suction range of thermocouple psychrometers usually is from 1 to 80 tons per square foot while the range of filter paper is from 0.1 to more than 1,000 tons per square foot. Two to seven days are required to reach moisture equilibrium for thermocouple psychrometer, while 7 days are required for filter paper. The thermocouple psychrometer method is simple and can be more accurate than filter paper after the equipment has been calibrated and the operating procedure established. The principal disadvantage is that the suction range is much more limited than the filter paper method, The filter paper method is technically less complicated than the thermocouple psychrometer method; however, the weighing procedure required for filter paper is critical and vulnerable to large error.

a. Calibration. The total soil suction is given on the basis of thermodynamics by the equation

$$\tau^\circ = -\frac{RT}{v_w} \ln \frac{p}{p_o} \qquad \text{(B-3)}$$

where

τ° = total suction free of external pressure except atmospheric pressure, tons per square foot
R = universal gas constant, 86.81 cubic centimetres-tons per square foot/mole-Kelvin
T = absolute temperature, Kelvin
v_w = volume of a mole of liquid water, 18.02 cubic centimetres/mole
p/p_o = relative humidity
p = pressure of water vapor, tons per square foot
p_o = pressure of saturated water vapor, tons per square foot

Equation (B-3) shows that the soil suction is related to the relative humidity in the soil. Both thermocouple psychrometer and filter paper techniques require calibration curves to evaluate the soil relative humidity from which the soil suction may be calculated using equation (B-3). Calibration is usually performed with salt solutions of various known molality (moles of salt per 1,000 grams of water) that produce a given relative humidity. Table B-1 shows the modalities re-

Table B-1. Calibration Salt Solutions

Measured temperature t, °C	Suction, tsf for cited molality of sodium chloride solution						
	0.053	0.100	0.157	0.273	0.411	0.550	1.000
15	3.05	4.67	7.27	12.56	18.88	25.29	46,55
20	3.10	4.74	7.39	12.75	19.22	25.76	47.50
25	3.15	4.82	7.52	13.01	19.55	26.23	48.44
30	3.22	4.91	7.64	13.22	19.90	26.71	49.37

quired for sodium chloride salt solutions to provide the soil suctions given as a function of temperature.

b. Thermocouple psychrometer technique. The thermocouple psychrometer measures relative humidity in soil by a technique called Peltier cooling. By causing a current to flow through a single thermocouple junction in the proper direction, that particular junction will cool, then water will condense on it when the dewpoint temperature is reached. Condensation of this water inhibits further cooling of the junction. Evaporation of condensed water from the junction after the cooling current is removed tends to maintain a difference in temperature between the thermocouple and the reference junctions. The microvoltage developed between the thermocouple and the reference junctions is measured by the proper readout equipment and related to the soil suction by a calibration curve.

(1) *Apparatus.* Laboratory measurements to evaluate total soil suction may be made with the apparatus illustrated in figure B-1. The monitoring system includes a cooling circuit with the capability of immediate switching to the voltage readout circuit on termination of the current (fig. B-2). The microvoltmeter (item 1, fig. B-2) should have a maximum range of at least 30 microvolt and allow readings to within 0.01 microvolt. The 12-position rotary selector switch (item 2) allows up to 12 simultaneous psychrometer connections. The 0-25 millimeter (item 3), two 1.5-volt dry cell batteries (item 4), and the variable potentiometer (item 5) form the cooling circuit, Equipment is available commercially to perform these measurements of soil suction.

(2) *Procedure.*

(a) Thermocouple psychrometer are inserted into 1-pint-capacity metal containers with the soil specimens, and the assembly is sealed with No. 13-1/2 rubber stoppers. The assembly is inserted into a 1- by 1- by 1.25-foot chest capable of holding six l-pint containers and insulated with 1.5 inches of foamed polystyrene, Cables from the psychrometer are passed through a 0.5-inch-diameter hole centered in the chest cover, The insides of the metal containers are coated with melted wax to inhibit corrosion of the containers.

(b) The apparatus is left alone until equilibrium is attained. Temperature equilibrium is attained within a few hours after placing the chest cover. Time to reach equilibrium of the relative humidity in the air measured by the psychrometer and the relative humidity in the soil specimen depends on the volume and initial relative humidity in the container. Equilibrium time may require up to 7 days, but may be reduced to 2 or 3 days by repeated testing of soils with similar suctions.

(c) After equilibrium is attained, the microvoltmeter is set on the 10- or 30-microvolt range and zeroed by using a zeroing suppression or offset control. The cooling current of approximately 8 millimeters is applied for 15 seconds and then switched to the microvoltmeter circuit using the switch of item 6 in figure B-2, The maximum reading on the microvoltmeter is recorded. The cooling currents and times should be identical to those used to determine the calibration curves.

(d) The readings can be taken at room temperature, preferably from 20 to 25 degrees Centigrade, and corrected to a temperature of 25 degrees Centigrade by the equation

$$E_{25} = \frac{E_t}{0.325 + 0.027_t} \qquad \text{(B-4)}$$

where

E_{25}= microvolt at 25 degrees Centigrade

E_t= microvolt at t degrees Centigrade

Placement of the apparatus in a constant temperature room will increase the accuracy of the readings.

(3) *Calibration,* The psychrometer are calibrated by placing approximately 50 millilitres of the salt solutions of known molality (table B-1) in the metal containers and following the procedure in b(2) above to determine the microvolt output. Equilibration time may be reduced to 2 or 3 days by surrounding the psychrometer with filter paper soaked with solution. The suctions given for the known modalities are plotted versus the microvolt output for a temperature of 25 degrees Centigrade. The calibration curves of 12 commercial psychrometer using the equipment of figure B-1 were within 5 percent and could be expressed by the equation

$$\tau_o = 2.65E_{25} - 1.6 \qquad \text{(B-5)}$$

where $\tau°$ is the total soil suction in tons per square foot. The calibration curves using other equipment may be somewhat different.

c. *Filter paper technique.* This method involves enclosing filter paper with a soil specimen in an airtight container until complete moisture equilibrium is

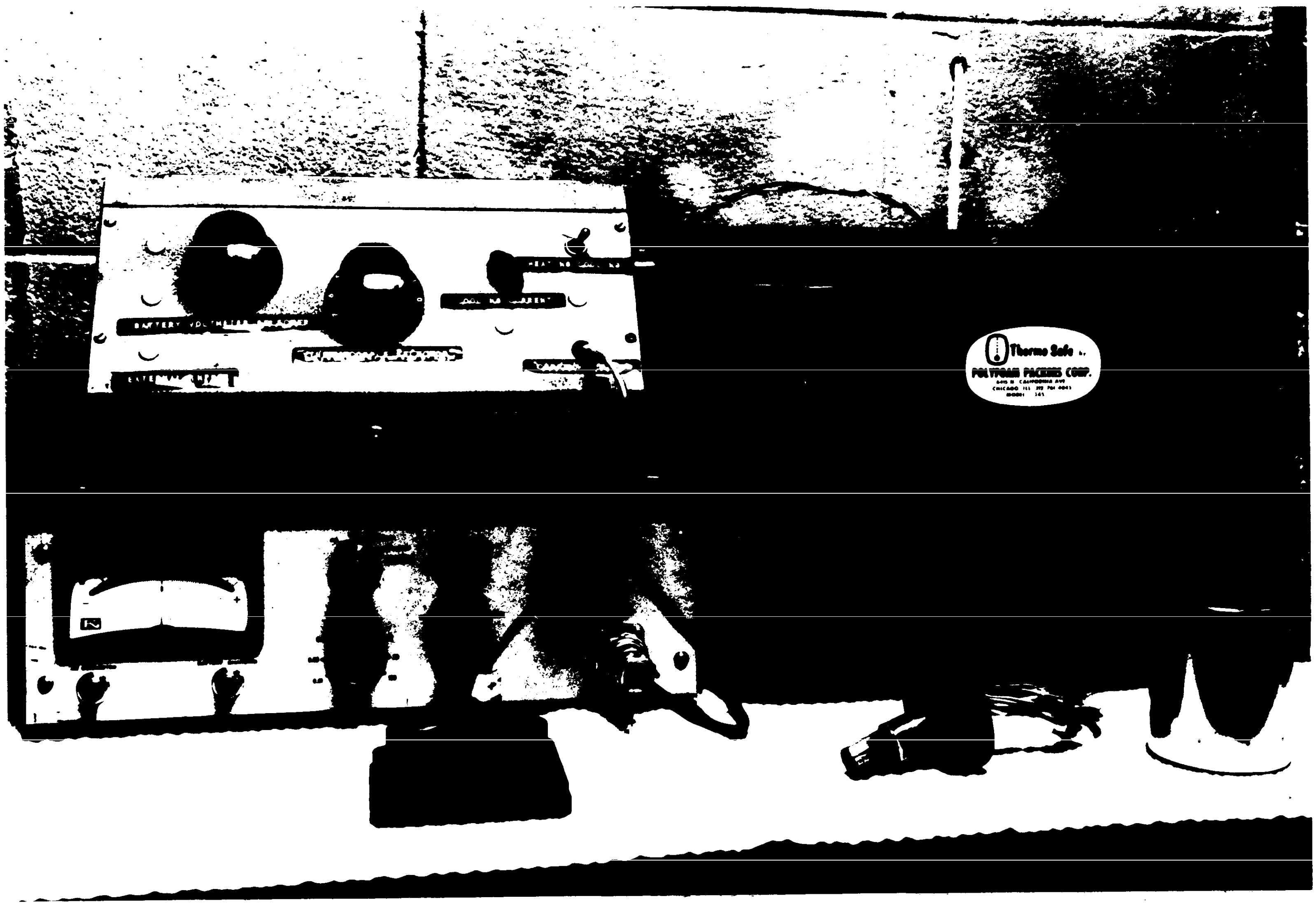

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Figure B-1. Thermocouple psychrometer monitoring apparatus.

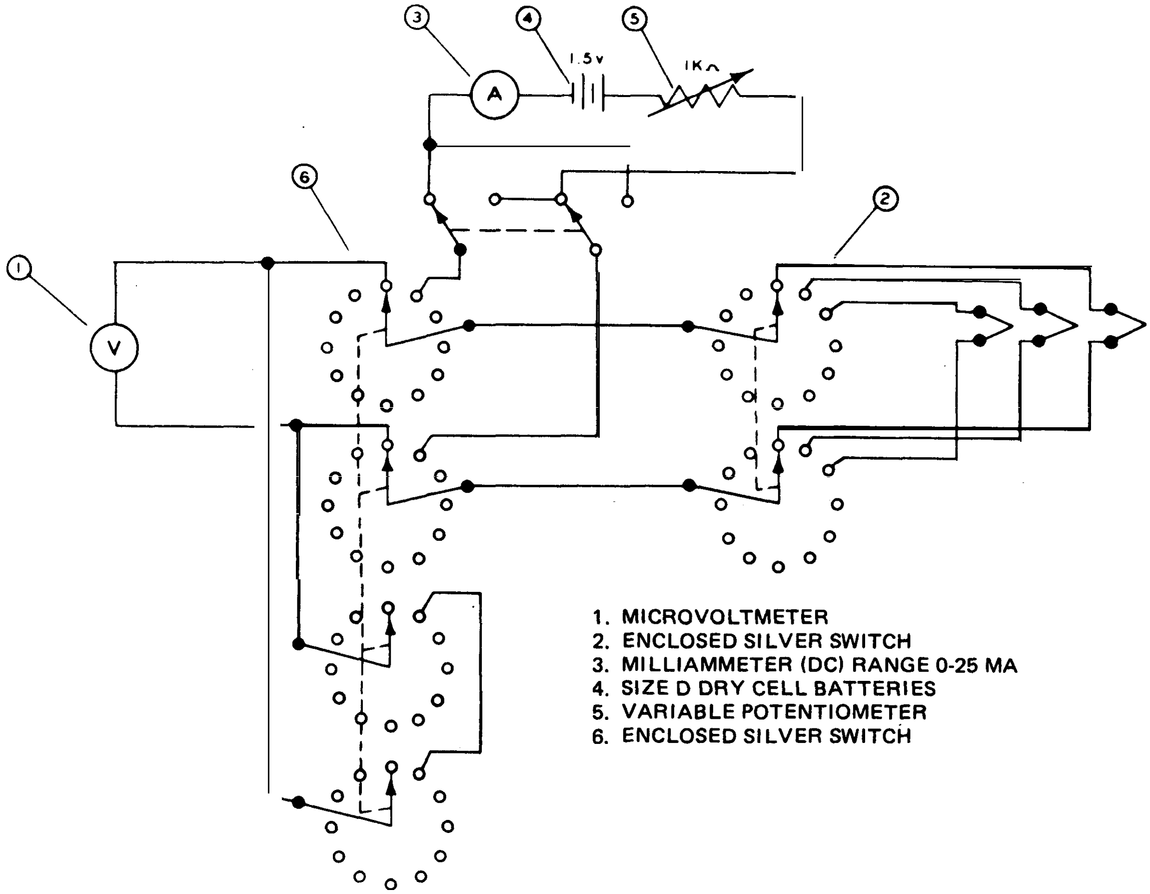

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Figure B-2. Electrical circuit for the thermocouple psychrometer.

reached. The water content in percent of the dry weight is subsequently determined, and the soil suction is found from a calibration curve.

(1) *Apparatus.* Materials consist of 2-inch-diameter filter paper, 2-inch-diameter tares, and a gravimetric scale accurate to 0.001 g. A filter paper is enclosed in an airtight container with the soil specimen.

(2) *Procedure.*

(a) The filter paper disc is pretreated with 3 percent reagent grade pentachlorophenol in ethanol (to inhibit bacteria and deterioration) and allowed to air dry. Reagent grade pentachlorophenol is required because impurities in the treatment solution influence the calibration curve. Care is required to keep the filter paper from becoming contaminated with soil from the specimen, free water, or other contaminant (e.g., the filter paper should not touch the soil specimen, particularly wetted specimens).

(b) Seven days are required to reach moisture equilibrium in the airtight container. At the end of 7 days, the filter paper is transferred to a 2-inch-cliameter covered tare and weighed immediately on a gravimetric scale accurate to 0.001 g. The number of filter papers and tares weighed at one time should be kept small (nine or less) to minimize error caused by water evaporating from the filter paper.

(c) The tare is opened and placed in an oven for **at least 4 hours or overnight at a temperature of 110 ±** 5 degrees Centigrade. The ovendry weight of the filter paper is then determined, and the water content as a percent of the dry weight is compared with a calibration curve to determine the soil suction.

(3) *Calibration.* The ovendry water content of the filter paper is dependent on the time lapse following removal from the drying oven before weighing.

(a) The calibration curves shown in figure B-3 were determined for various elapsed times following removal from the oven. The calibrations are given for Fisherbrand filter paper, Catalog Number 9-790A, enclosed with salt solutions of various molality for 7 days. Calibration curve No. 1 resulted from weighing the filter paper 5 seconds following removal from the oven. Time lapses of 15 minutes and 4 hours lead to a similar calibration curve (No. 3) of significantly smaller water contents than the 5-second curve for identical suctions. Calibration curve No. 2 was determined

by removing 12 specimens from the oven, waiting 30 seconds to cool, then weighing as soon as possible and within 15 minutes.

(b) Calibration curves based on the method used to determine curve No, 3 with a waiting time between 15 and 30 minutes are recommended if the suctions of

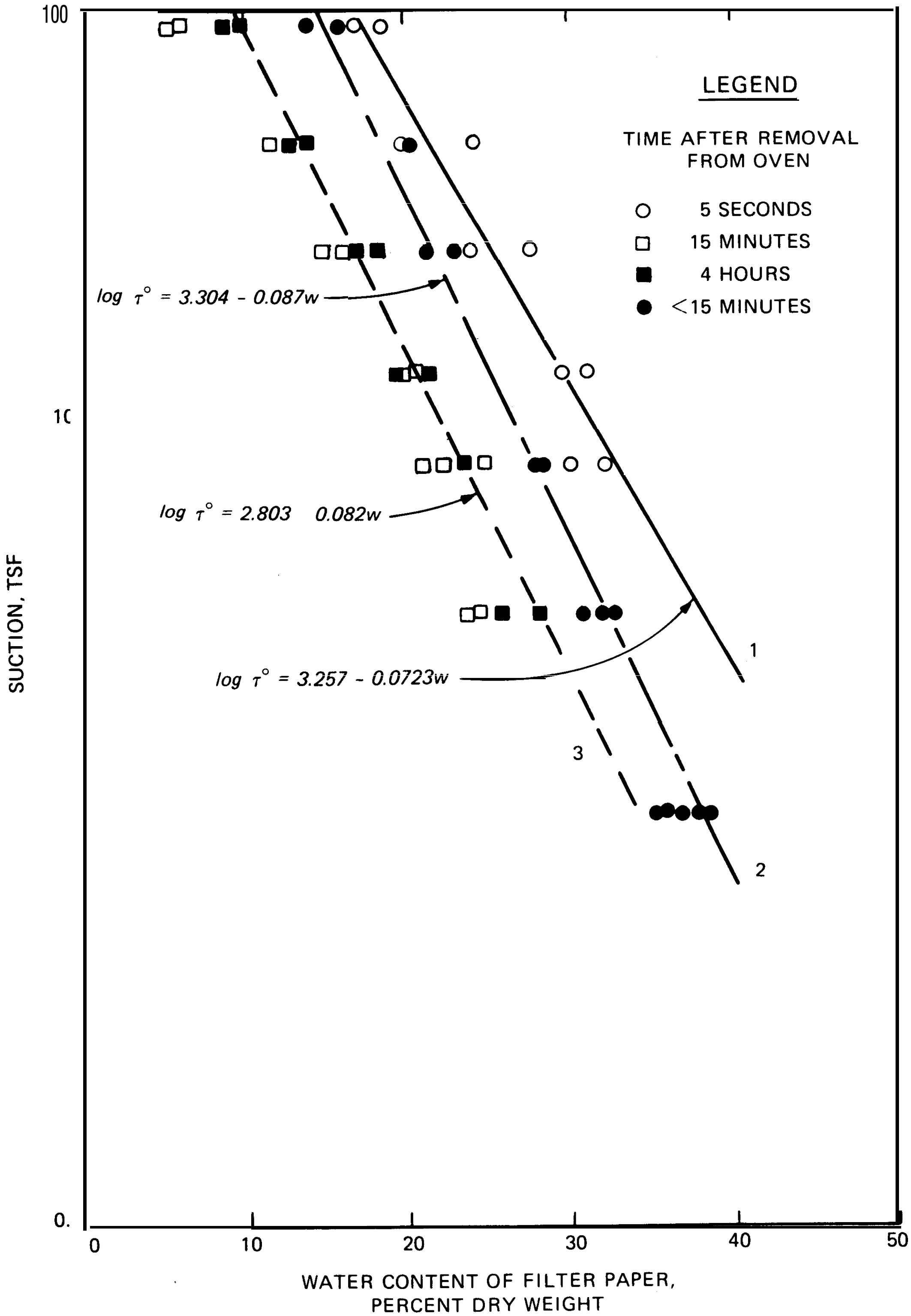

Figure B-3. Calibration of filter paper.

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large numbers of specimens are to be evaluated. However, the accuracy will be less than if curve No. 1 and its procedure are used because curve No. 3 can be influenced by changes in the relative humidity of the testing room. The gravimetric scale should be located near the drying oven for the 5-second calibration curve (No. 1) to be practical. Changes in filter paper weights are normally small (e.g., less than 0.1 g) and require accurate calibration of the gravimetric scale and adherence to a single standardized procedure.

B-3. Characterization of swell behavior

The swell behavior of a particular soil may be characterized from the matrix suction-water content relationship and the compressibility factor α to calculate heave by the equation

$$\frac{\Delta H}{H} = \frac{e_1 - e_o}{1 + e_o} = \frac{C_\tau}{1 + e_o} \log \frac{\tau^o_{mo}}{\tau^o_{mf}} \quad \text{(B-6)}$$

where

ΔH = potential vertical heave at the bottom of the foundation, feet
H = thickness of the swelling soil
e_1 = final void ratio following swell
e_o = initial void ratio
C_τ = $\alpha G_s/100B$, suction index
α = compressibility factor
G_s = specific gravity
B = slope soil suction parameter
τ^o_{mo} = initial matrix suction without surcharge pressure, tons per square foot
τ^o_{mf} = final matrix suction without surcharge pressure, tons per square foot

The suction index C_τ is similar to $\gamma_h(1 + e_o)$ where γ_h is the suction compression index of the McKeen-Lytton method in table 4-2. Equation (B-6) is similar to equation (5-2) of paragraph 5-4*a* and equation (5-8) of paragraph 5-4*a* from which the total potential heave is calculated. Equation (B-6) will also lead to the same or similar predictions of heave for identical changes in suction. The suction index, a measure of the swelling capability, is analogous to the swell index c_s of consolidometer swell tests, except that the suction index is evaluated with respect to the change in matrix suction without surcharge pressure rather than the change in effective pressure.

a. Matrix suction and water content relationship. This relationship is evaluated from the total soil suction and water content relationship. The total soil suction as a function of water content is found from multiple 1-inch pieces of the undisturbed sample. The pore water may be evaporated at room temperature, for various periods of time up to about 48 hours, from several undisturbed specimens; various amounts of distilled water may also be added to several other undisturbed specimens of each sample to obtain a multipoint water content distribution. Each specimen may be inserted into a 1-pint metal container with a thermacouple psychrometer or with filter paper to evaluate the total soil suction as previously described. The dry density and void ratio of each undisturbed specimen from which the compressibility factor α is determined may be evaluated by the water displacement method. Using thermocouple psychrometers, collect soil suction data on DA Form 5182-R (Soil Suction, Water Content and Specific Volume). DA Form 5182-R will be reproduced locally on 11- by 8½-inch paper. A copy of DA Form 5182-R for local reproduction purposes can be found at the back of this manual.

(1) The multipoint total soil suction and water content relationship may be plotted as shown in figure B-4 for each undisturbed sample. The open circles in the figure represent natural water content w_o, and the closed circles symbolize water being added to or evaporated from the undisturbed specimens at room temperature. An osmotic suction τ_s is sometimes indicated by a horizontally inclined slope at high water contents, and the magnitude may be estimated by noting the total soil suction at high water contents. Large osmotic suctions appreciably flatten the slope as shown in figure B-4. The matrix suction and water content relationship can be approximated by subtracting the osmotic suction from the total soil suctions and expressing the result as

$$\log \tau^o_m = A - Bw \quad \text{(B-7)}$$

where

τ^o_m = matrix suction without surcharge pressure, tons per square foot
A = ordinate intercept soil suction parameter, tons per square foot
B = slope soil suction parameter
w = water content, percent dry weight

Information on piezometric pore water pressures is used in approximating the matrix suction and water content relationship in the presence of appreciable osmotic suctions.

(2) The matrix suction and water content relationship of figure B-4 was approximated by noting that the groundwater elevation, at which $u_w = 0$, was 1.5 feet. Hence, the matrix suction at the natural water content of 27 percent was the total mean confining pressure σ_m of approximately 0.1 ton per square foot from equation (B-1). The value σ_m may be estimated from equation (B-2) if K_T can be approximated. The remainder of the curve was approximated by subtracting 26 tons per square foot, which was the total average suction at the natural water content of 27 percent less 0.1 ton per square foot, from the total soil suction observed at smaller water contents. The osmotic suction should be subtracted from the total suction; otherwise heave predictions will be overestimated since the osmotic suction does not appear to cause much heave and

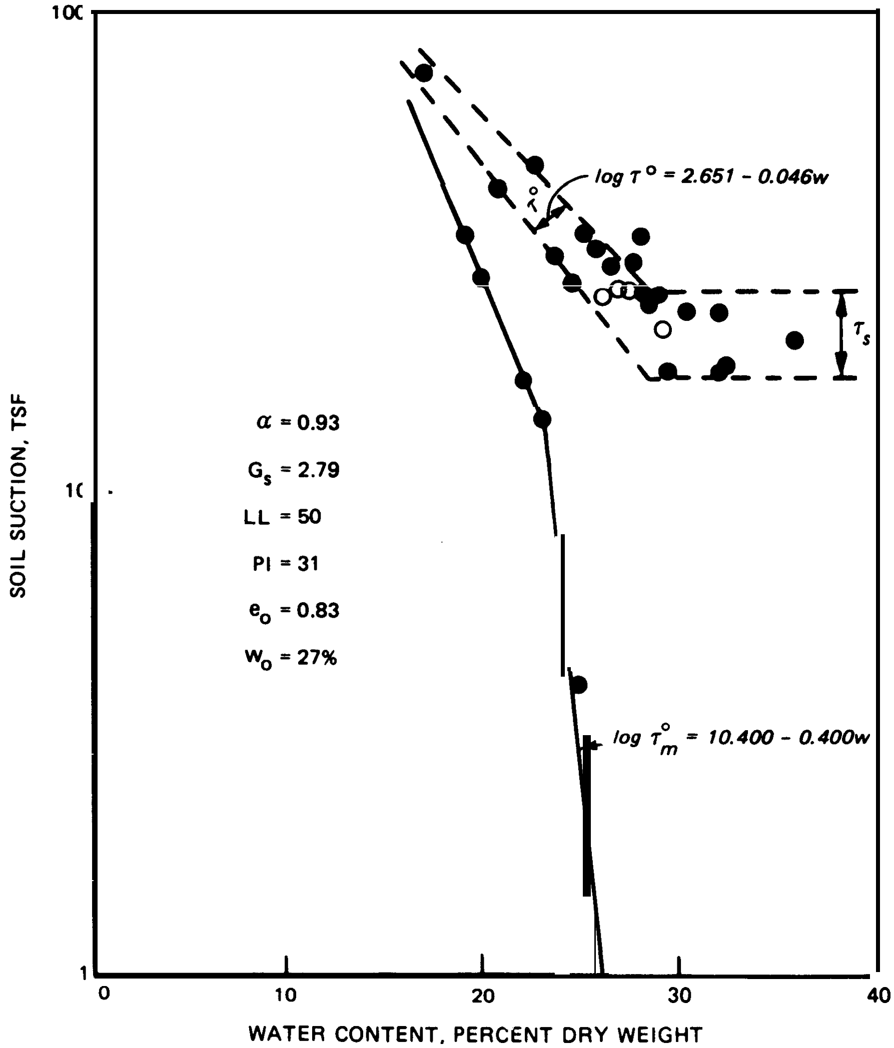

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Figure B-4. Soil suction and water content relationship for Fort Carson overburden at 1 to 3 feet of depth.

if the equilibrium moisture profiles of figure 5-1 (para 5-4b) are used.

b. Initial matrix suction. The initial matrix suction *τ&without* surcharge pressure may be evaluated using the soil suction test procedure on undisturbed specimens or may be calculated from equation (B-7) and the natural (initial) water content.

c. Final matrix suction. The final matrix suction τ°_{mf} without surcharge pressure may be calculated from the assumption

$$\tau^{\circ}_{mf} = \left(\frac{1 + 2K}{3}\right) \sigma'_v \qquad \text{(B-8)}$$

K = coefficient of effective lateral earth pressure

σ'_v = final vertical effective pressure, tons per square foot or from equation (B-1) setting a = 1 and if K_T can be approximated.

The final vertical effective pressure may be found from

$$\sigma'_v = \sigma_v - u_w \qquad \text{(B-9)}$$

where σ_v is the final total vertical pressure. The pore water pressure u_w (fig. 5-1) is found from equations (5-3), (5-4), or (5-5).

d. Compressibility factor. The compressibility factor *a* is the ratio of the change in volume for a corresponding change in water content, i.e., the slope of the curve γ_w/γ_d plotted as a function of the water content where γ_w is the unit weight of water and γ_d is the dry density. The value of *a* for highly plastic soils is close to 1, and much less than 1 for sandy and low plasticity soils. High compressibility y factors can indicate highly swelling soils; however, soils with all voids filled with water also have a equal to 1.

(1) Figure B-5 illustrates the compressibility factor calculated from laboratory data for a silty clay taken from a field test section near Clinton, Mississippi. Extrapolating the line to zero water content, as shown in the figure, provides an estimate of l/R with

$$R = \frac{W_s}{V_o} \qquad \text{(B-10)}$$

where

R = shrinkage ratio
W_s = mass of a specimen of ovendried soil, grams
V_o = volume of a specimen of ovendried soil, cubic centimetres

(2) The shrinkage limit SL of the clay shown in figure B-5 may be taken as the abrupt change in slope of the curve, which is 23.3 percent. The SL is calculated by the following equation:

$$SL = w - \frac{V - V_o}{W_s} \times 100 \qquad \text{(B-11)}$$

where w is the water content and V is the volume of the wet soil specimen in cubic centimetres. Equation (B-11) assumes that $\alpha = 1$. For soils with α less than 1, the SL varies depending on the initial water content of the specimen. For example, if the initial water content is at the natural water content of 25.7 percent, then equation (B-11) will give

$$SL = 25.7 - (0.658 - 0.588)\,100 = 18.7 \qquad \text{(B-12)}$$

as shown in figure B-5. Other shrinkage limits may be evaluated by drawing straight lines with slope $\alpha = 1$ through other water content points. Soils with the PI less than 40 are more likely to indicate compressibility factors less than 1 than higher plasticity soils. Equation (B-11) is not applicable to soils with α much less than 1.

e. Examples.

(1) The potential heave of the soil characterized by figure B-4 may be calculated from equation (B-6). The final in situ pore water pressure u_w is equal to 0 at the groundwater level of 1.5 feet. If the depth H is 1.5 feet, then σ_v = 0.09 ton per square foot. From these variables and the parameters in DA Form 5182-R.

$$C_\tau = \frac{\alpha G_s}{100B} = \frac{(0.93)(2.79)}{(100)(0.400)} = 0.065$$

$$\tau^{\circ}_{mo} = 10^{10.400-0.400w_o} = 0.398 \text{ ton per square foot}$$

$$\tau^{\circ}_{mf} = u_w + \alpha\sigma_v = 0 + 0.93\,(0.09) = 0.084 \text{ ton per square foot}$$

Therefore,

$$\frac{\Delta H}{H} = \frac{C_\tau}{1 + e_o} \log \frac{\tau^{\circ}_{mo}}{\tau^{\circ}_{mf}}$$

$$= \frac{0.065}{1 + 0.83} \log \frac{0.398}{0.084} = 0.024$$

The potential heave ΔH will be 0.036 foot or 0.4 inch for the 1.5-foot layer of soil overburden. Practically, the computation indicates that ½ inch of heave is expected.

(2) If the osmotic component of suction is not known, then the potential heave may still be roughly approximated by noting that the mean minimum total suction at high water content is 22 tons per square foot in the example of figure B-4. This value may be taken as the final total soil suction τ°_f. The initial value of total soil suction τ°_o is found by noting that the mean total soil suction at natural water content is 26 tons per square foot in figure B-4. The slope B of the total soil suction and water content curve is subsequently used to evaluate the suction index C_τ. The potential heave for this case will be

$$CT = \frac{(0.93)\ (2.79)}{(100)\ (0{,}046)} = 0.564$$

$$\frac{\Delta H}{H} = \frac{C_\tau}{1 + e_o} \log \frac{\tau^{\circ}_o}{\tau^{\circ}_f}$$

$$= \frac{0.564}{1 + 0.83} \log \frac{26}{22} = 0.022$$

The potential heave ΔH will be 0.033 foot or 0.4 inch for the 1.5-foot layer of soil overburden. The disadvantage of this latter approach is that the equilibrium matrix suction or pore water pressure profile is not known, except that the final matrix suction will be small and probably close to the saturated profile (equation (5-3)). The program HEAVE will compute the potential heave for this case as well as those shown in figure 5-1.

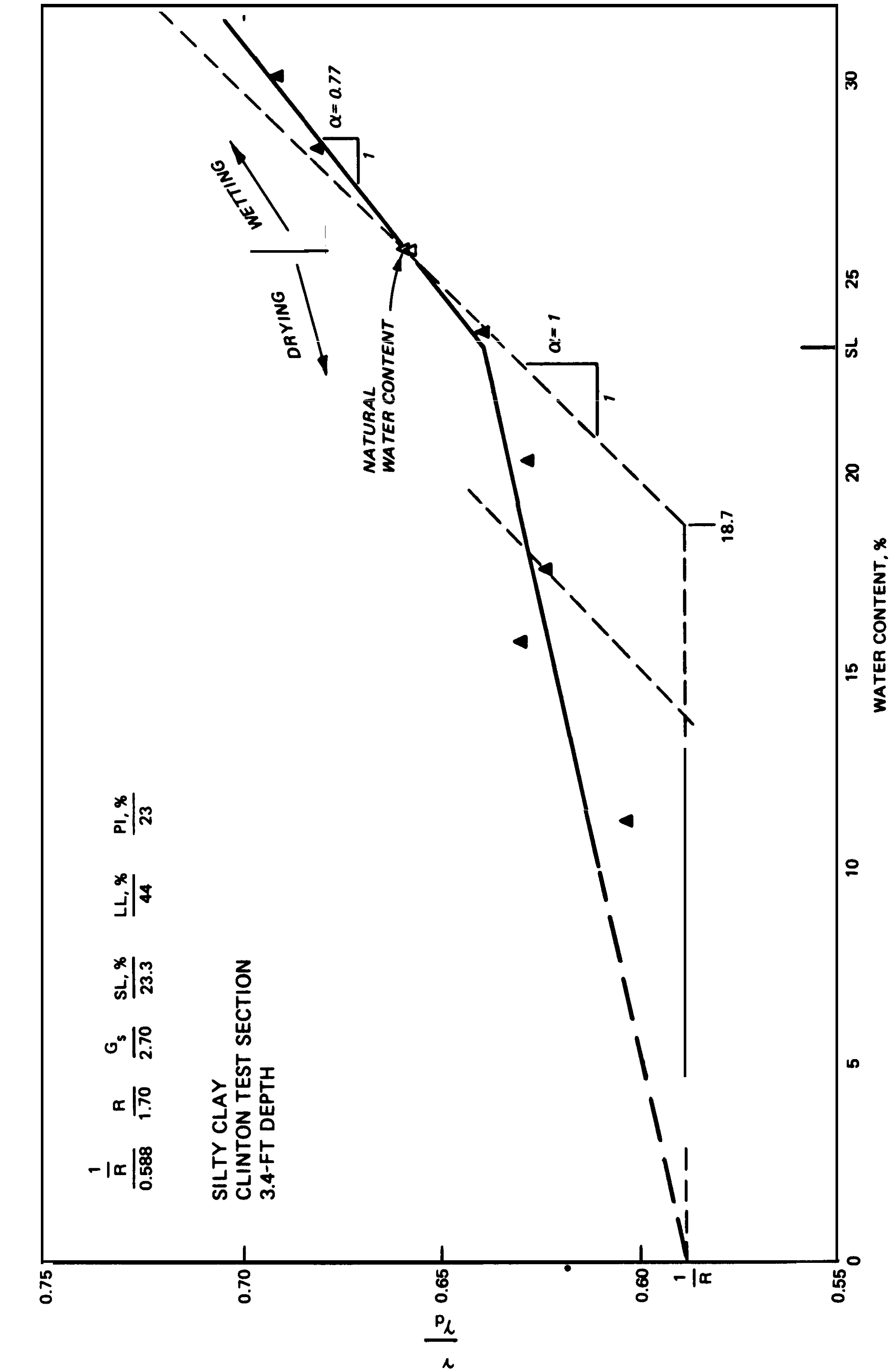

Figure B-5. Illustration of the compressibility factor.

APPENDIX C

FRAME AND WALL CONSTRUCTION DETAILS

Figures C-1 through C-10 illustrate types of construction for expansive foundation soils. These figures were taken from U.S. Army Corps of Engineers Construction Engineering and Research Laboratory Technical Report M-81. The figures show practical wall ties to concrete and steel beams, wall connections with control joints, details of interior partitions, bar joist first floor framing with grade beams, and stiffened mat foundations.

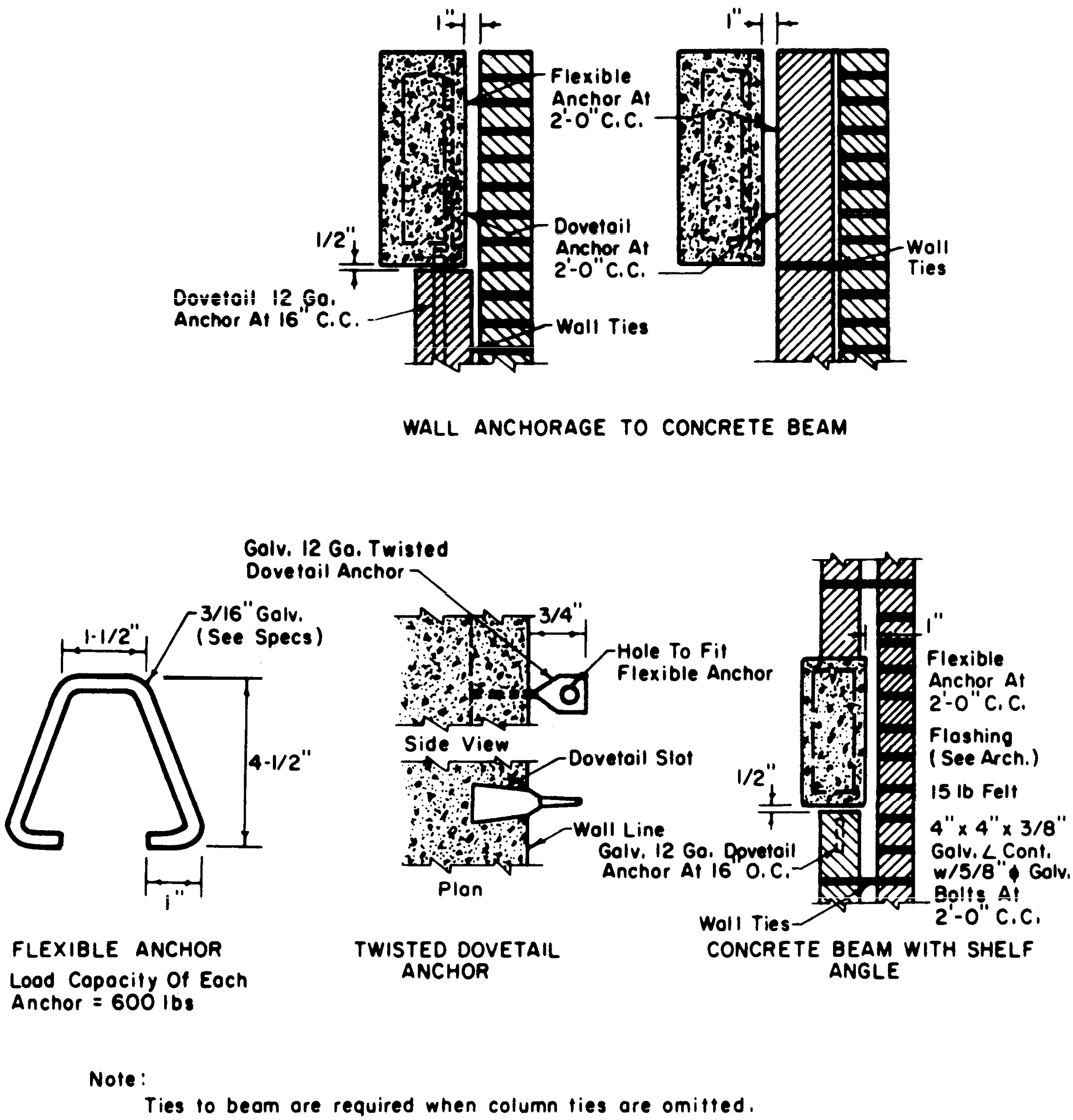

Figure C-1. Wall ties to concrete beams.

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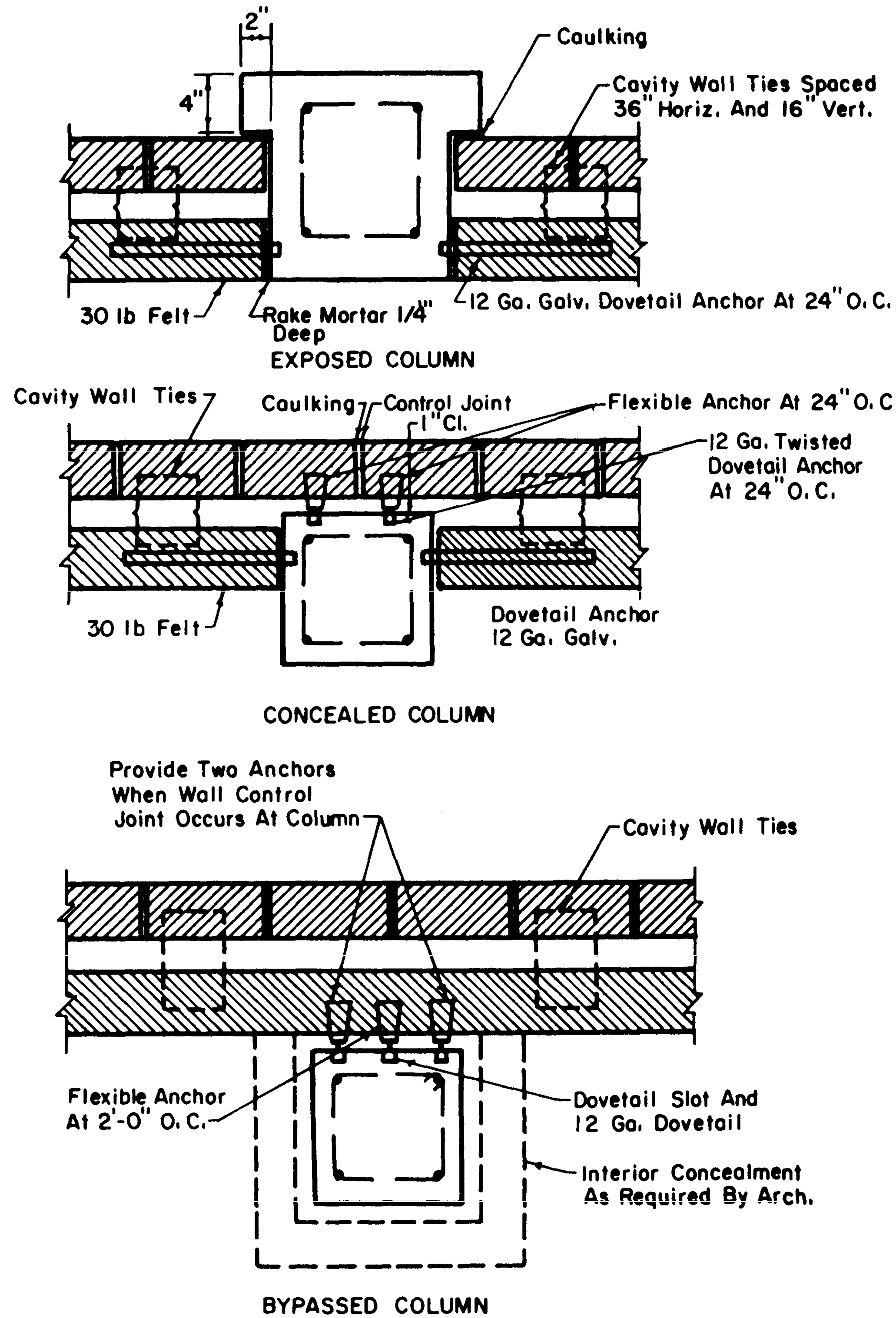

Note: Ties to column are required only when ties to beam are omitted.

Figure C-2. Wall ties to concrete column.

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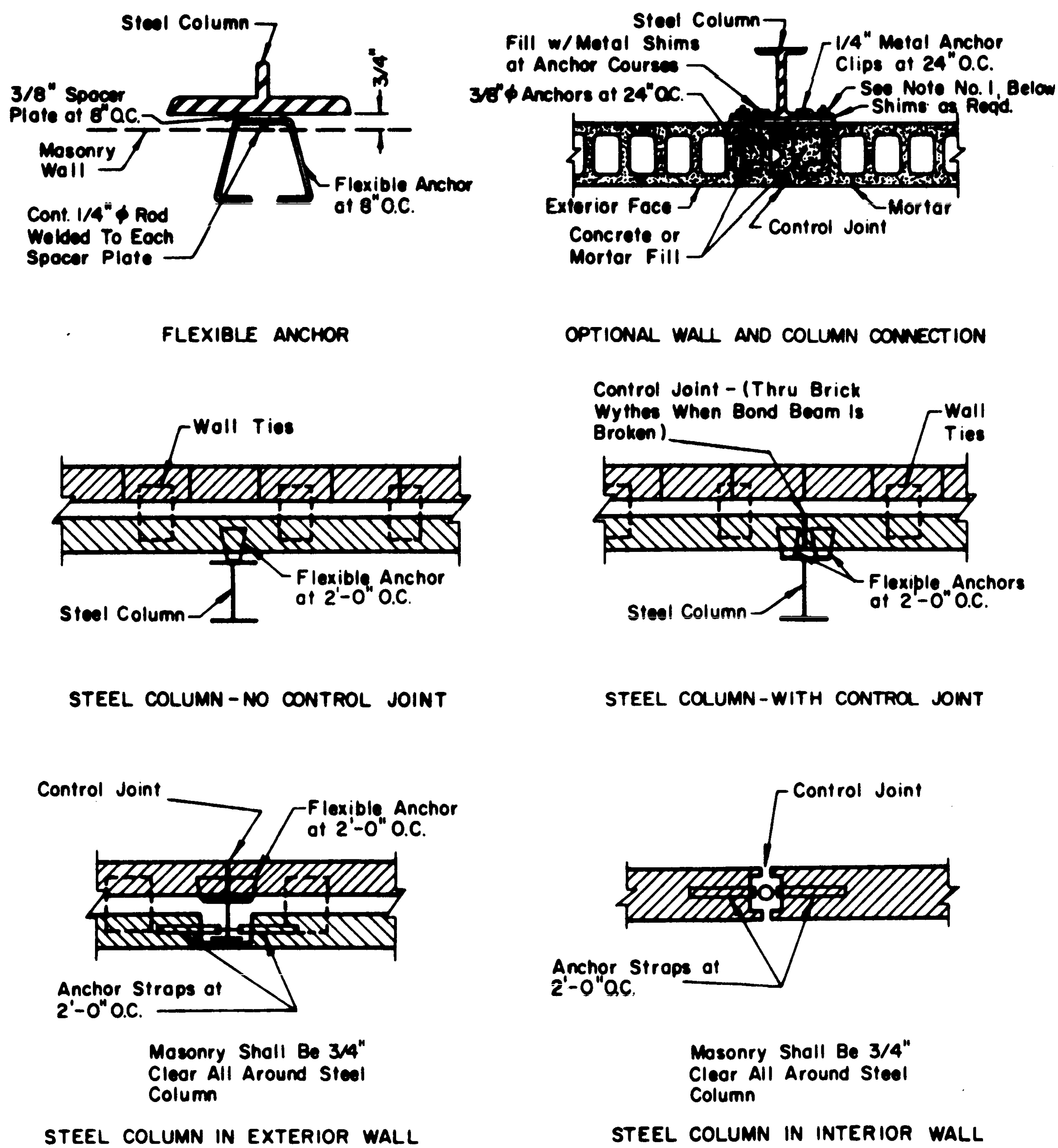

Ties To Columns Are Required Only When Ties To Beam Above Are Omitted.
Do Not Connect Column To Wall At Corners of Buildings

Note:

1. Nuts Should Not Be Tightened Excessively, Horizontal Movement of Wall Is Necessary.

Figure C-3. Wall ties to steel column (Sheet 1 of 2).

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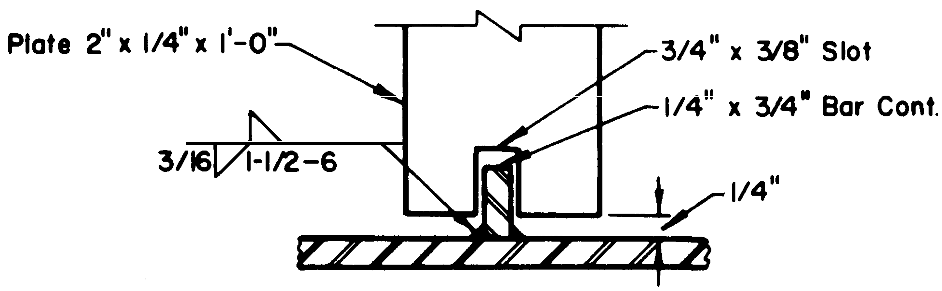

DETAIL "A"

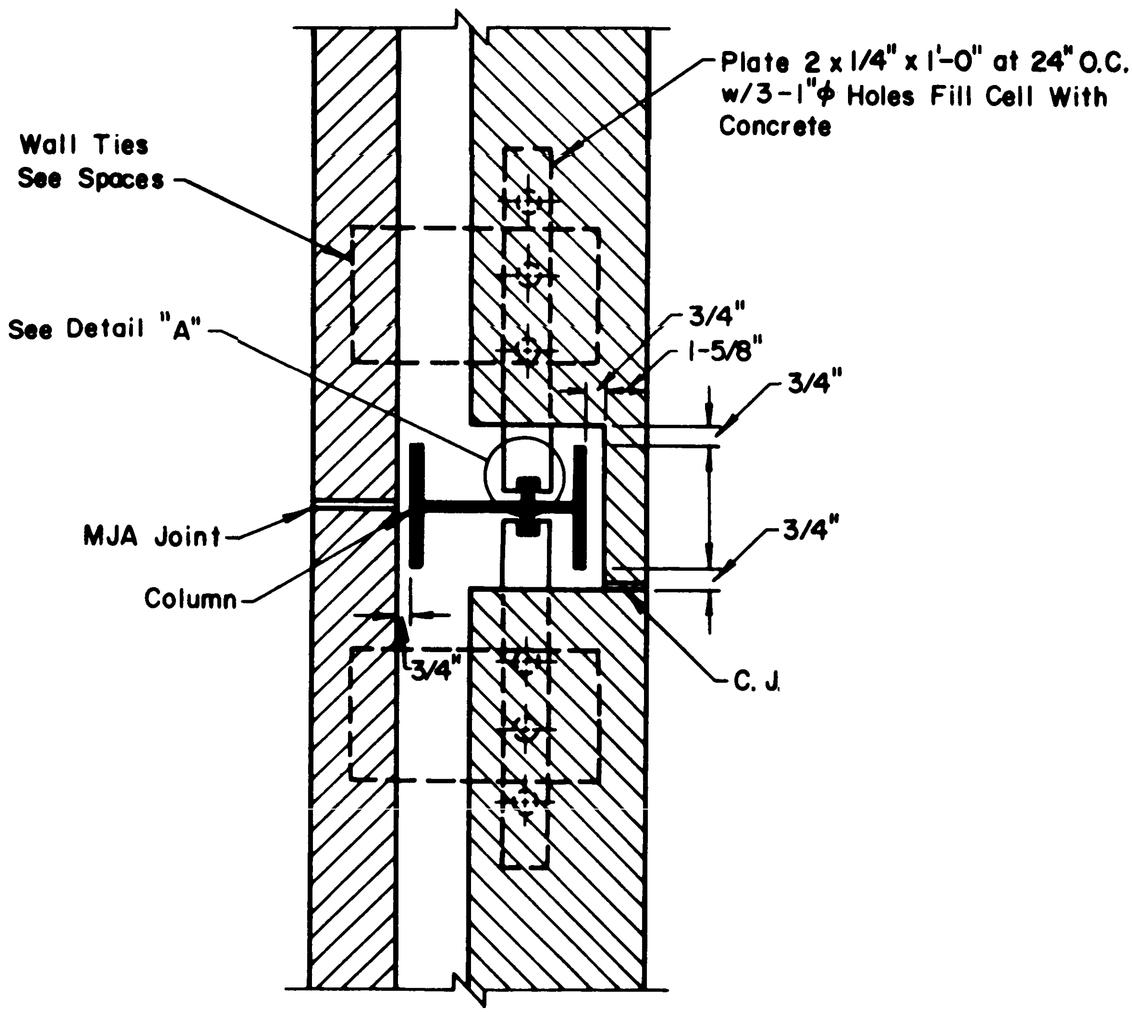

Figure C-3. (Sheet 2 of 2).

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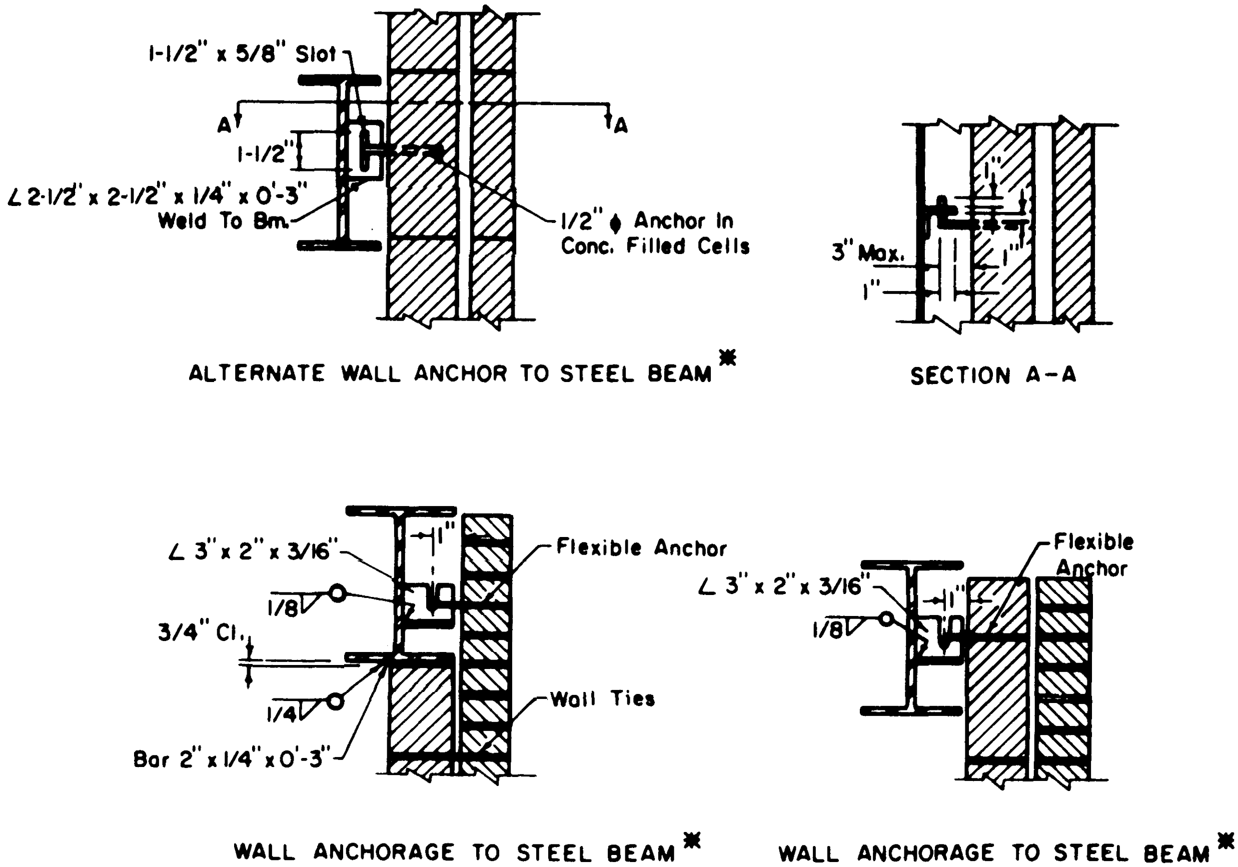

Note:

Ties to beam required only when ties to column are omitted.

* 2'-0" Spacing for Exterior Walls
4'-0" Spacing for Interior Walls

Figure C-4. Wall ties to steel beam.

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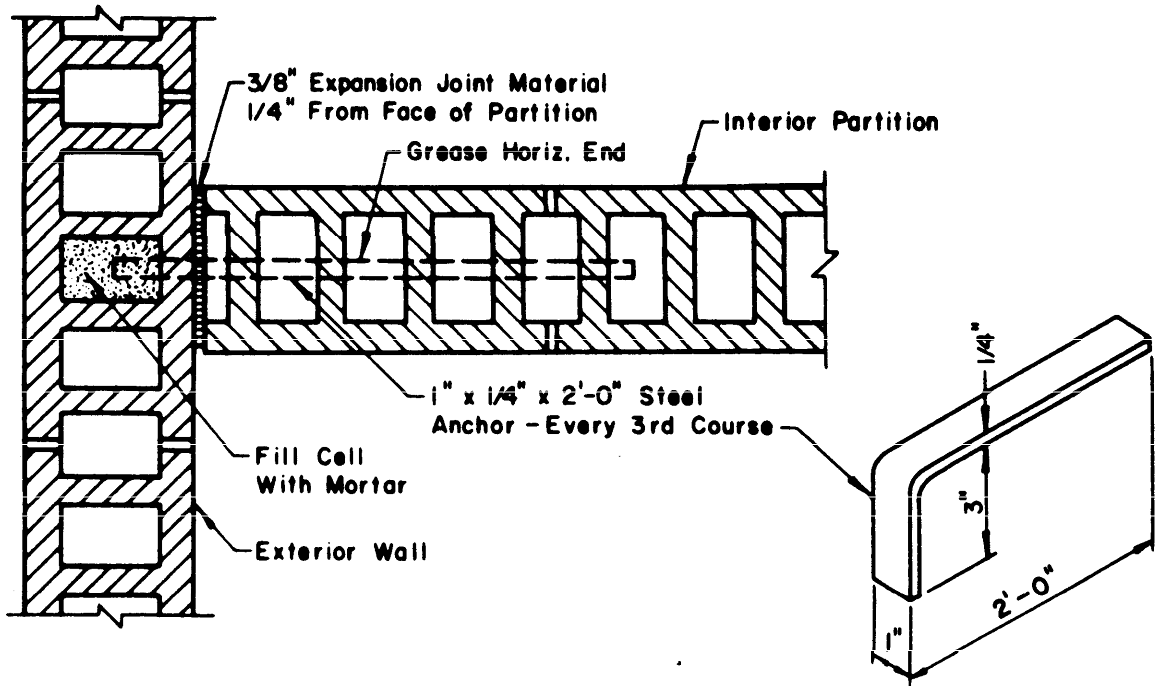
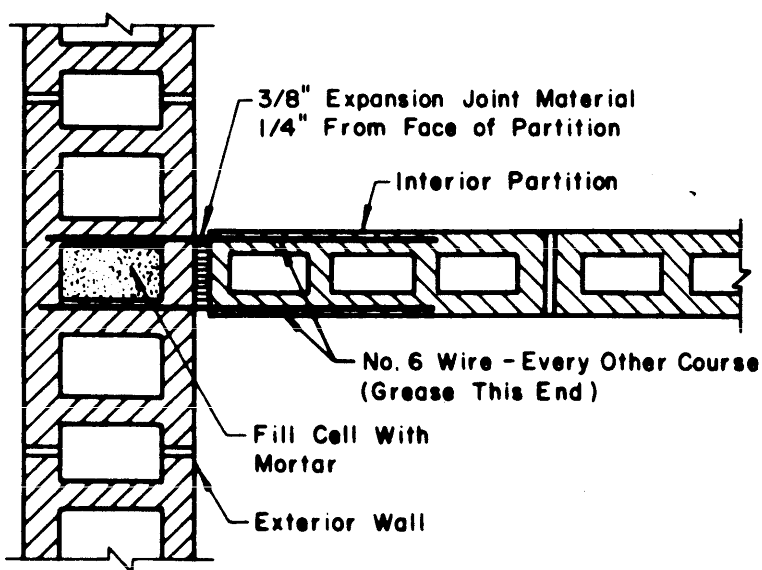


FOR PARTITIONS 6" WIDE OR WIDER



FOR 4" WIDE PARTITIONS

Figure C-5. Wall connections with control joints.

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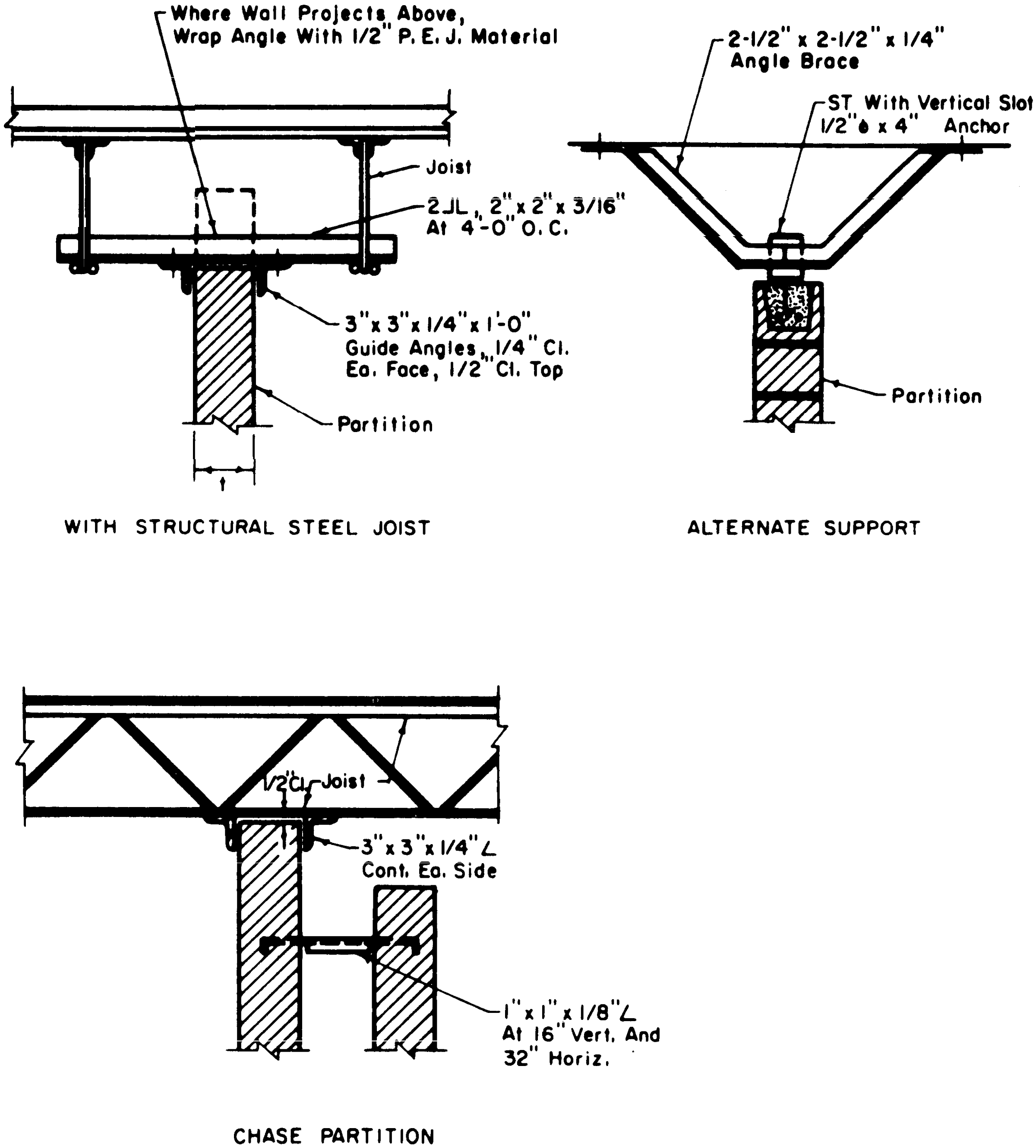

Figure C-6. Typical details of interior partitions.

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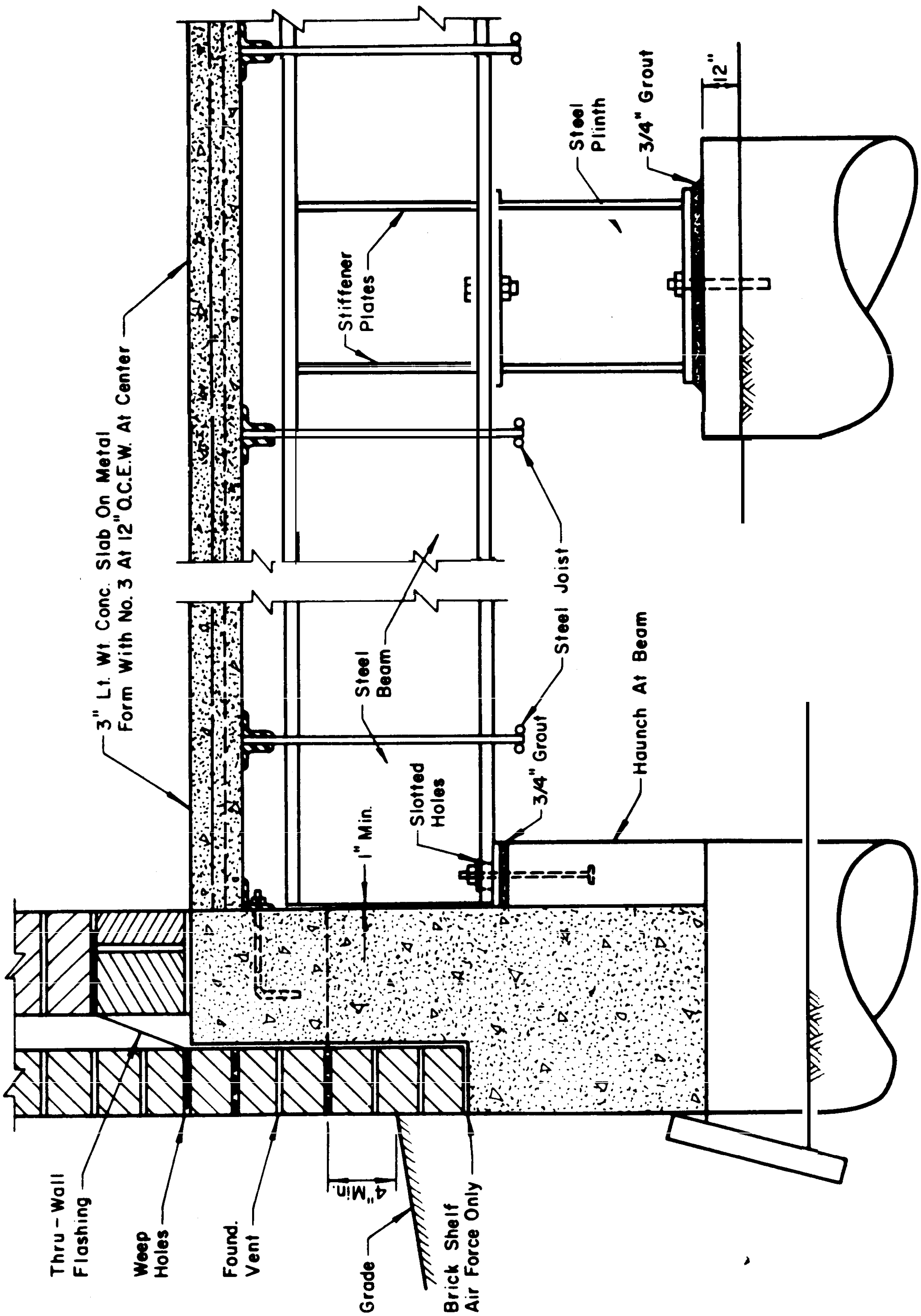

Figure C-7. Typical bar joist first floor framing.

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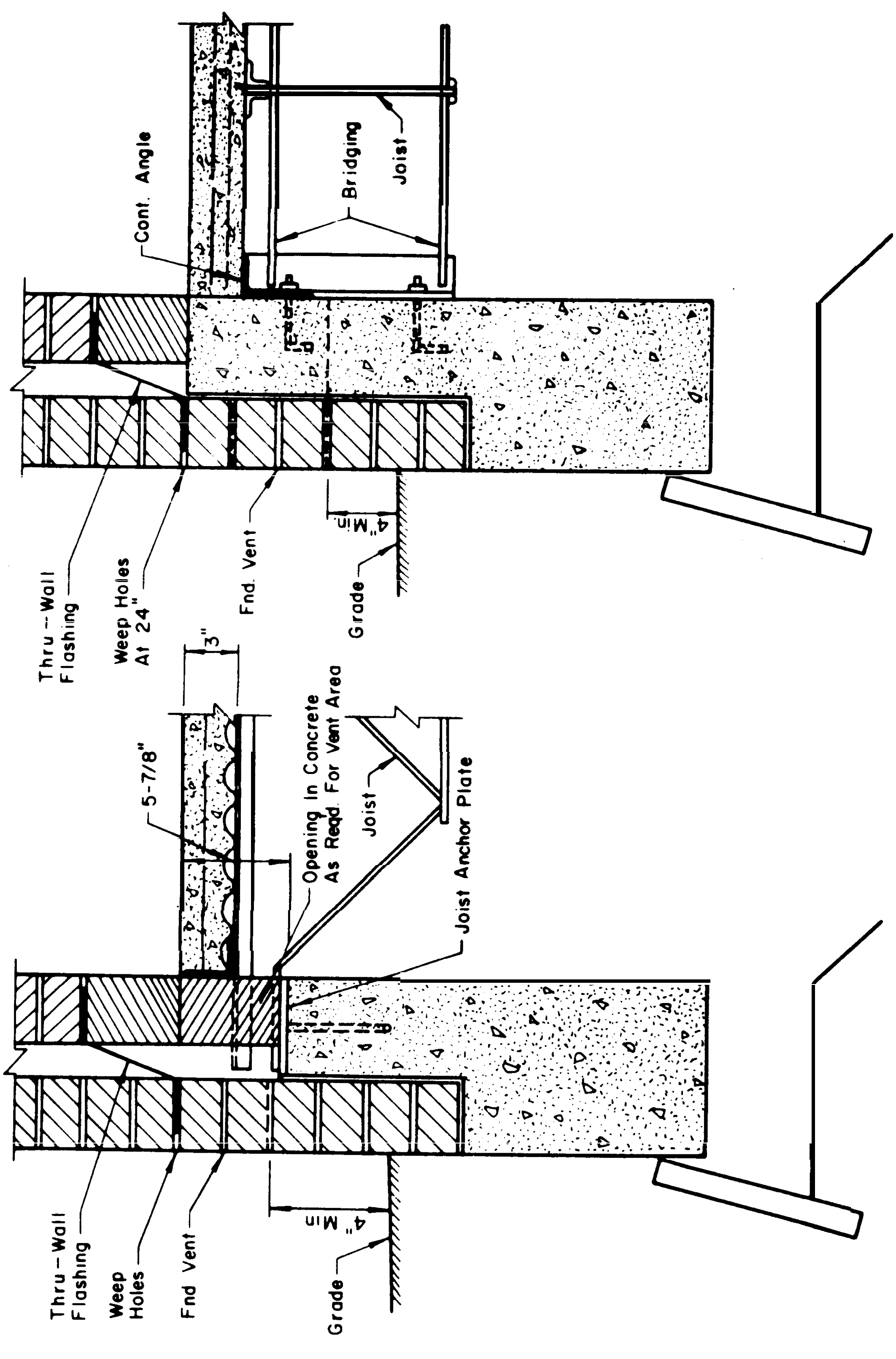

Fig C-8. Typical cast-in-place or precast concrete grade beam with steel bar joist floor framing.

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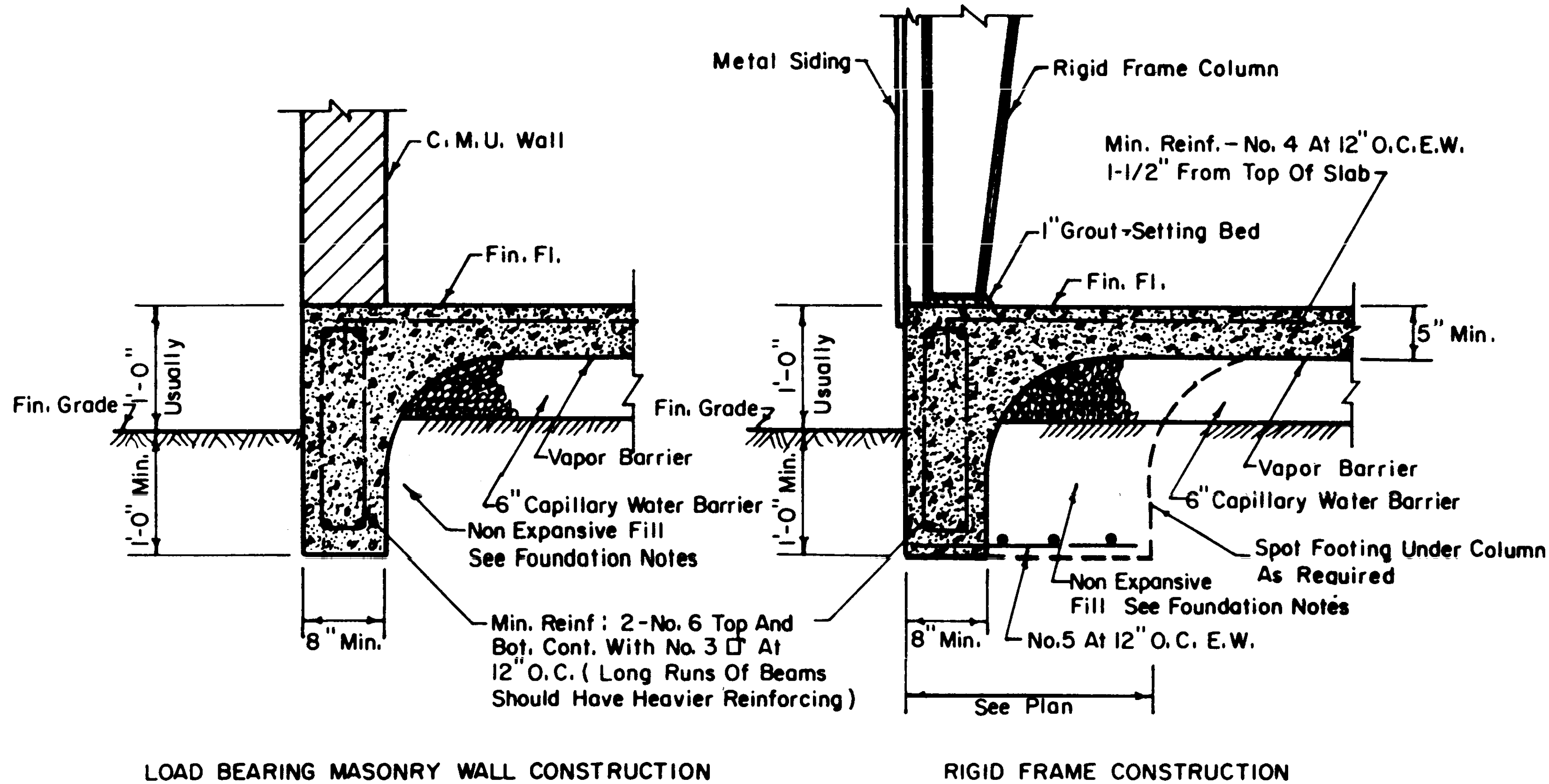

Figure C-9. Typical ribbed mat foundations.

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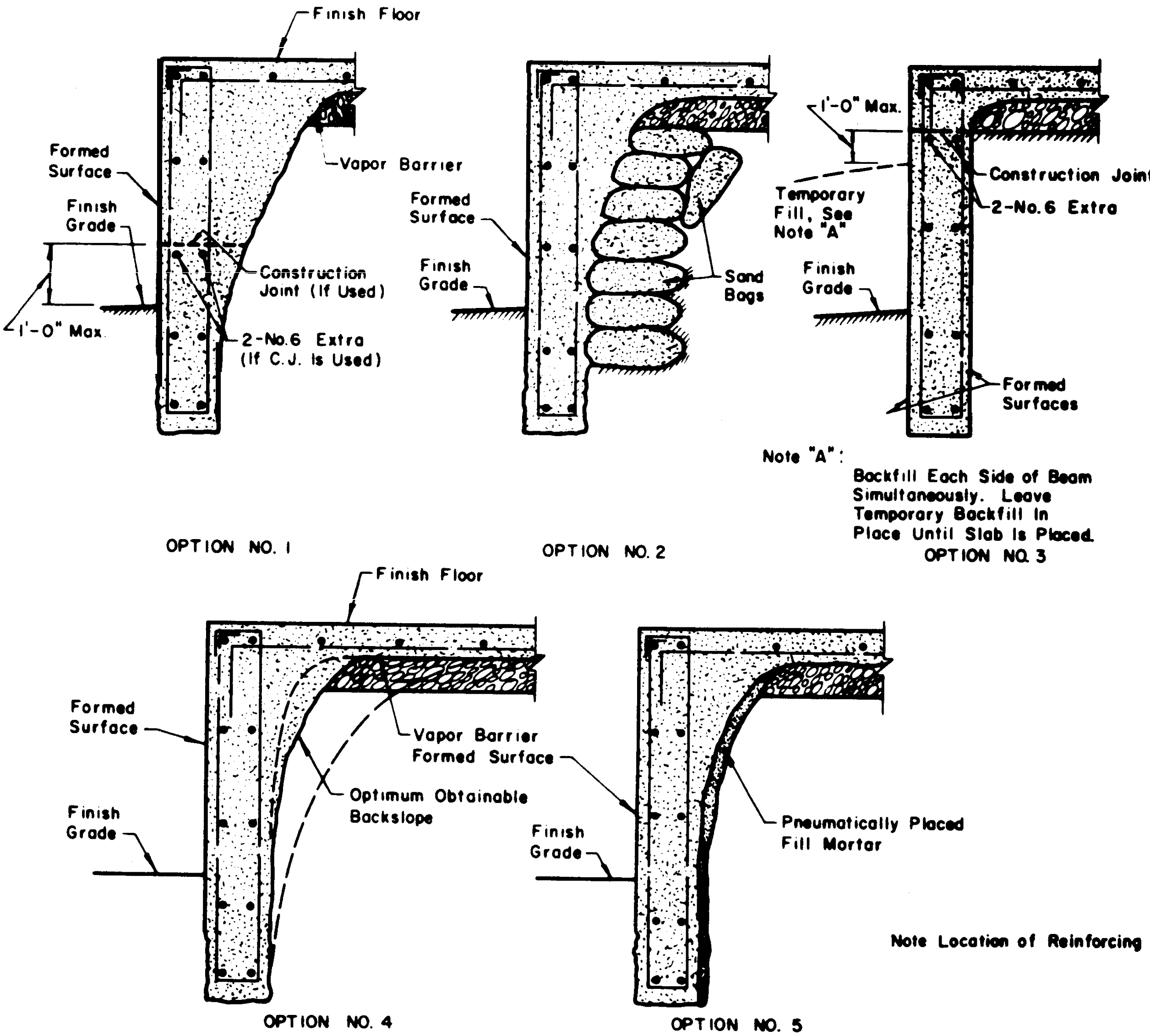

Figure C-10. Optional construction details of exterior beams (interior beam similar) for ribbed mat construction.

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APPENDIX D

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PROJECT ______ BORING/SAMPLE/DEPTH ______ DATE ______

SOIL SUCTION	PSYCHROMETER NO.												
	SAMPLE CONTAINER NO.												
	WATER CONTENT INCREMENT (0, +, -)												
	THERMOCOUPLE OUTPUT	t, MILLIVOLTS											
		T; °C											
	PSYCHROMETER OUTPUT	ET, MICROVOLTS											
		$E^{**}_{25°C}$, MICROVOLTS											
	SOIL SUCTION†, TONS/FT^2		τ										
WATER CONTENT	TARE NO.												
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		TARE PLUS DRY SOIL											
		WATER	W_w										
		TARE											
		DRY SOIL	W_s										
	WATER CONTENT, PERCENT		w										
WEIGHT-VOLUME RELATIONS	TEST TEMPERATURE OF WATER, °C												
	WEIGHT IN GRAMS	WET SOIL AND WAX IN AIR											
		WET SOIL	W										
		WAX											
		WET SOIL AND WAX IN WATER											
		DRY SOIL ††	W_s										
	SPECIFIC GRAVITY OF SOIL		G_s										
	VOLUME IN CC	WET SOIL AND WAX ‡											
		WAX											
		WET SOIL	V										
		DRY SOIL = W_s/G_s	V_s										
	DENSITY PCF	WET DENSITY = (W/V) 62.4	γ_m										
		DRY DENSITY = (W_s/V) 62.4	γ_d										
	VOID RATIO = $(V - V_s)/V_s$		e										
	POROSITY, % = $[(V - V_s)/V] \times 100$		n										
	DEGREE OF SATURATION, % = $[V_w/(V - V_s)] \times 100$		S										
	SPECIFIC VOLUME = $1/\gamma_d$		V_T										

* $T°C = t/0.0395$

** $E_{25} = E_T/(0.325 + 0.027T)$

† SEE INDIVIDUAL PSYCHROMETER CALIBRATION LINE

†† IF NOT MEASURED DIRECTLY, MAY BE COMPUTED AS FOLLOWS: $W_s = \frac{W}{1 + 0.01\,w}$

‡ VOLUME OF WET SOIL AND WAX = $\frac{\text{(WEIGHT OF WET SOIL AND WAX IN AIR)} - \text{(WEIGHT OF WET SOIL AND WAX IN WATER)}}{\text{DENSITY OF WATER AT TEST TEMPERATURE}}$

VOLUME OF WAX = $\frac{\text{WEIGHT OF WAX}}{\text{SPECIFIC GRAVITY AT WAX}}$

DA FORM 5182-R, MAR 83

Foundations in Expansive Soils Part 2
Updated on: 10/22/2012

1. _________ significantly influence the magnitude and rate of foundation movement.

    a) Groundwater
    b) Climate
    c) Vegetable cover
    d) all of the above

2. A desirable reliability is that the predicted potential total vertical heave should not be less than _________ of the maximum insitu heave that will eventually occur but should not exceed the maximum insitu heave by more than 20-50%.

    a) 20%
    b) 40%
    c) 80%
    d) 95%

3. Differential heave results from edge effects beneath a finite covered area and _________.

    a) drainage patterns
    b) lateral variations in thickness of the expansive foundation soil
    c) effects of occupancy
    d) all of the above

4. Swelling of expansive foundation soils should be considered during the _________ design phase and the level of structural cracking that will be acceptable to the user should be determined at this time.

    a) final
    b) preliminary
    c) post
    d) none of the above

5. Stiffened mat foundations are applicable in swelling soil areas where predicted differential movement AH may reach _________.

    a) 1 inch
    b) 2 inches
    c) 4 inches
    d) 6 inches

6. The flexibility required to avoid undesirable distress may be provided by joints and _________ connections.

    a) friction
    b) moment
    c) flexible
    d) all of the above

7. Structures supported by __________ footings are susceptible to damages from lateral and vertical movement of foundation soil is provisions are not made to accommodate possible movement.

   a) pile
   b) shallow individual
   c) mat
   d) all of the above

8. Basements and long continuous footings constructed in excavations are subject to swell pressures from underlying and adjacent expansive soil.

   a) True
   b) False

9. Concrete slabs without internal __________ are much more susceptible to distortion or doming from heaving soil.

   a) control joints
   b) stiffening beams
   c) slip joints
   d) fiber reinforcement

10. Concrete mats for heavy structures tend to be __________ or more in thickness with a continuous two-way reinforcement top and bottom.

    a) 1 foot
    b) 3 feet
    c) 7 feet
    d) 10 feet

11. The design and construction of drilled shaft foundations must be closely controlled to avoid distress and damage. Most problems have been caused by defects in construction and by inadequate design considerations for effects of __________.

    a) high water tables
    b) swelling soil
    c) shrinkage of soil
    d) none of the above

12. __________ resistance develops from small relative displacements between the shaft and the adjacent soil.

    a) Tip
    b) Skin
    c) Shear
    d) Swelling

13. Skin resistance may also be evaluated in terms of effective stress from results of drained direct shear tests.

    a) True
    b) False

14. Grade beams spanning between shafts are designed to support wall loads imposed vertically downward. These grade beams should be isolated from the underlying swelling soil with a void space beneath the beams of 6 to 12 inches or _________ times the predicted total heave of soil located above the base of the shaft foundation (whichever is larger).

    a) 0.50
    b) 1.50
    c) 2.00
    d) 5.00

15. Construction in new excavations (within a few years of excavating) without replacement of surcharge pressure equal to the original soil overburden pressure should be avoided where possible because the reduction in effective stress leads to an instantaneous elastic rebound plus a time-dependent heave.

    a) True
    b) False

16. Two effective and most commonly used soil stabilization techniques are controlled backfilling and continuous maintenance involving drainage control and limited watering of surface soil adjacent to the structure during droughts.

    a) True
    b) False

17. In general, the natural soil should be compacted to _________ of standard maximum density and should be wet of optimum water content.

    a) 70%
    b) 80%
    c) 90%
    d) 100%

18. A _________ membrane moisture barrier can be used to promote uniform soil moisture beneath the foundation by minimizing the loss or gain of moisture through the membrane and thus reducing cyclic edge movement.

    a) plastic
    b) asphalt
    c) granular material
    d) all of the above

19. __________ treatment may be applied to minimize downhill soil creep of slopes greater than 5 degrees (9%) by increasing the stiffness and strength of the soil mass through filling fractures in the surface soils.

   a) Lime
   b) Asphalt
   c) Slurry
   d) Bismuth

20. Damage to drilled shafts is often caused by __________ movement of the shaft from swelling soil beneath its base and by uplift forces on the shaft perimeter from adjacent swelling soil.

   a) upward
   b) downward
   c) sideways
   d) frictional