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Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive and Stable Soils

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POST-TENSIONING INSTITUTE
Strength in Concrete

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RECOMMENDATIONS

1.0 — SCOPE

This standard is intended to be used by licensed design professionals (LDPs) and provides minimum requirements for the design of shallow post-tensioned concrete foundations on expansive and stable soils. Internal forces and stiffness requirements specified in this standard shall be used for design of all ribbed and uniform-thickness post-tensioned foundations built on soils that satisfy the criteria specified in Section 8.1.

COMMENTARY

C1.0 — SCOPE

This combined standard, incorporating both geotechnical and structural standards into a single document, is based on principles of unsaturated soil mechanics for predicting support conditions, internal forces, and stiffness requirements affecting shallow concrete foundations built on and interacting with expansive soils. Additionally, this standard applies to post-tensioned slabs on stable soils.

Shallow post-tensioned concrete foundations are commonly used in single-family and multi-family residential, light commercial, and low-rise commercial construction.

The following foundation types are defined:

- PTI-1: Lightly reinforced slabs on stable soils. These slabs may be post-tensioned to eliminate joints required in unreinforced slabs and/or to control shrinkage and temperature cracking (which can occur before the tendons are stressed), and load transfer, in accordance with the provisions of this standard (Section 4.1.3). These slabs are usually built on stable soils.
- PTI-2: Reinforced and stiffened slabs on expansive soils
- PTI-3: Uniform thickness slabs on expansive soils

The soil-structure interaction codified herein is applicable to all shallow foundations built on expansive soils, regardless of the type of reinforcement (prestressed or non-prestressed), within the limitations stated herein.

Design methods for concrete foundations on expansive soils, which yield smaller values of internal forces and stiffness requirements than those specified in this standard (PTI-2 and PTI-3 slabs), may result in inadequate foundation strength and underestimation of foundation stiffness.

This combined standard does not address compressible, or collapsible soils. Post-tensioned foundations can be used for these types of soils by using other rational design methods.

Post-tensioned concrete foundations designed by this standard generally meet the requirements for plain concrete specified in Chapter 14 of ACI 318-14.[1] These foundations will typically contain less reinforcement—prestressed and non-prestressed—than the ACI 318 requirements for reinforced concrete. This standard is intended to be a

RECOMMENDATIONS

2.0 — DEFINITIONS AND ABBREVIATIONS

2.1 — Definitions

Edge drop – a soil-structure distortion mode wherein the soil moisture content at the perimeter of the foundation is typically lower than the soil moisture content beneath the center of the foundation. Alternatively referred to as center lift (Fig. C2.1).

Edge lift – a soil-structure distortion mode wherein the soil moisture content at the perimeter of the foundation is typically higher than the soil moisture content beneath the center of the foundation (Fig. C2.1).

COMMENTARY

stand-alone document uniquely developed for the design of post-tensioned concrete foundations on expansive and stable soils and is supported by the performance of many thousands of existing conformant foundations. As such, it is intended that this standard be independent of ACI 318 and the conflicting parts of the general building code into which this standard is incorporated.

This standard is based on PTI DC10.1-08.[2] Refer to this document and the commentary to this standard for background and interpretational information that clarifies its application.

C2.0 — DEFINITIONS AND ABBREVIATIONS

C2.1 — Definitions

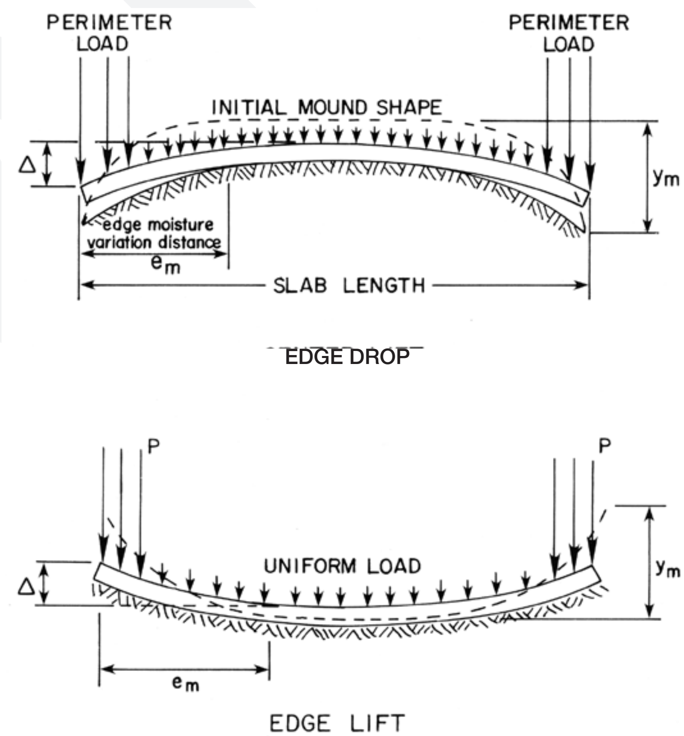

Fig. C2.1—Edge drop and edge lift.

RECOMMENDATIONS

Licensed design professional (LDP) – design professional licensed in the state in which they are practicing and qualified in the area under their responsible charge.

Noncompliant rectangle – a rectangle which can be mathematically generated from a slab geometry but which does not include the properties to be either a primary design rectangle or secondary design rectangle.

Post-construction suction envelope – a design envelope that assumes the foundation is constructed when the soil at the site may be in a condition of extreme dryness from a prolonged dry period or extreme wetness from a prolonged wet period.

Post-equilibrium suction envelope – a design envelope that assumes the foundation is constructed when the soil at the site will likely be in a condition near or at equilibrium.

Primary design rectangle – a design rectangle encapsulating the most contiguous portions of the foundation which represents the largest portion of the foundation and has congruency in both directions and includes the maximum perimeter boundary conditions practical (Fig. C2.2).

COMMENTARY

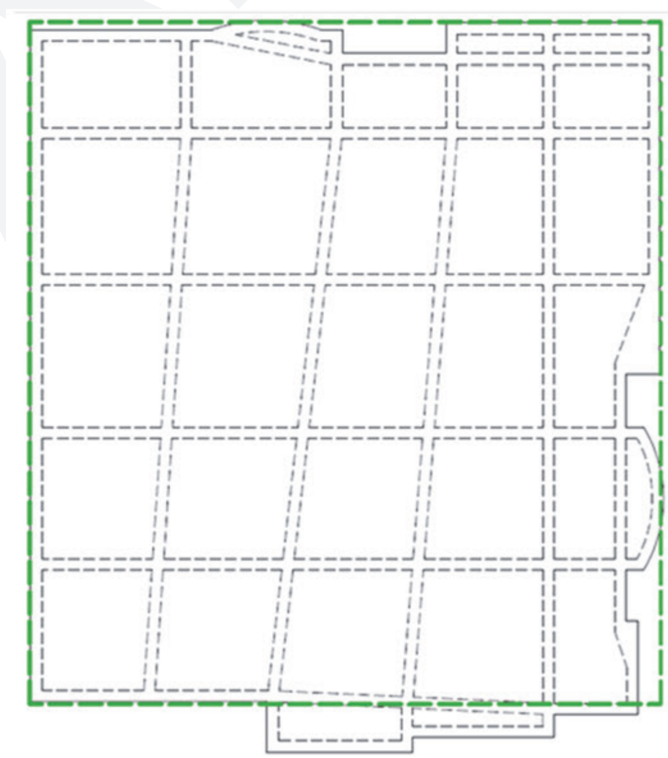

Fig. C2.2 — Primary design rectangle example.

A given design may include multiple primary design rectangles.

RECOMMENDATIONS

Ribbed foundation – a foundation system consisting of a uniform thickness slab with ribs that satisfy the requirements of Section 4.2.2 and project downward from the bottom of the slab in both directions. The slab and ribs are considered to act monolithically.

Secondary design rectangle – a design rectangle which includes specific portions of the foundation which extend outside the limits of the primary design rectangle (Fig. C2.3).

COMMENTARY

Primary design rectangles may include small sections of void within the continuity.

A portion of the primary design rectangle may exist outside the footprint.

The primary design rectangle should provide reasonably accurate moments in both directions based upon the aspect ratio of the true footprint of the foundation.

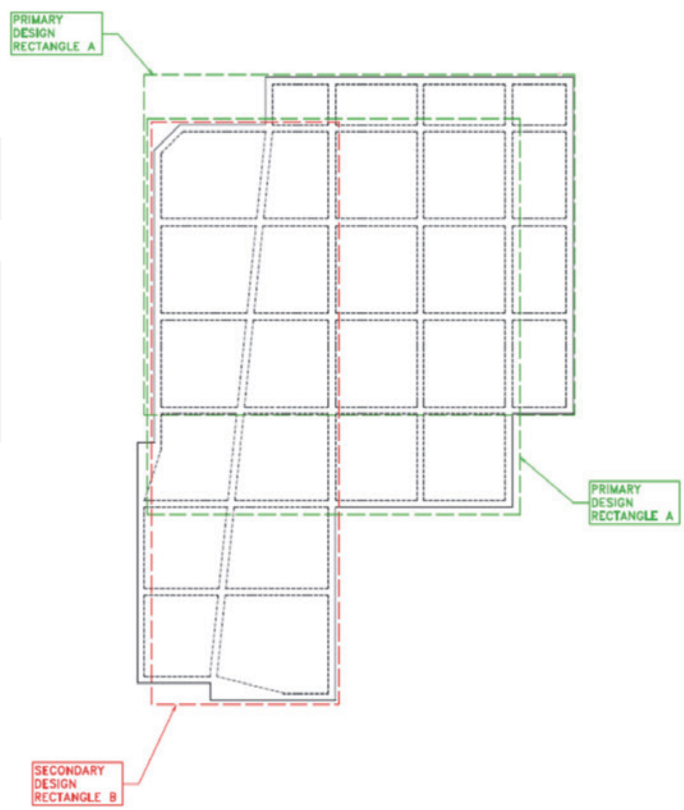

Fig. C2.3 – Secondary design rectangle example.

Secondary design rectangles are not required for small projections from the primary design rectangles, when deemed structurally insignificant by the licensed design professional.

The licensed design professional should use sound engineering judgment as well as past experience on the design of the interface of these appendages.

RECOMMENDATIONS

Stiffness – for purposes of this standard, product of E_{cr} and I.

Uniform thickness foundation (UTF) – a foundation system consisting of a solid slab of uniform thickness with no interior ribs.

2.2 — Abbreviations

CGC = geometric centroid of gross concrete section

CGS = center of gravity of prestressing force

COMMENTARY

3.0 — NOTATION

C3.0 — NOTATION

Equations in this standard are unit-specific—that is, variables must be entered with units specified in this section.

Sign convention used for force or stress throughout this standard is tension (negative) and compression (positive). Moments are positive if producing tension at the bottom of the foundation and negative if producing tension at the top of the foundation.

Unless specifically stated otherwise, all foundation parameters (geometry, internal forces, prestress force, reinforcement, and so on) are based on the entire cross section or full width of the section being designed.

A = area of gross concrete cross section in direction being considered, in.^{2}

A_b = bearing area beneath tendon anchor, in.^{2}

A_b' = maximum area of portion of bearing surface that is geometrically similar to and concentric with tendon anchor, in.^{2}

A_{bm} = total area of rib concrete = nbh, in.^{2}

A_o = coefficient in equation for M_L

A_{ps} = total cross-sectional area of prestressed reinforcement, in.^{2}

A_s = total cross-sectional area of non-prestressed reinforcement, in.^{2}

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A_{sl} = total cross-sectional area of slab concrete, in.^{2}

A_v = area of rib shear reinforcement, in.^{2}

B = constant used in equation for M_L

B_w = assumed slab width, in.

b = width of individual rib, in.

C = constant used in equation for M_L

C_s = coefficient to increase or decrease the required stiffness based on building materials and their reaction to movement

CR = prestress loss due to creep of concrete, kip

c = distance between CGC and extreme cross section fibers, in.

E_c = modulus of elasticity of concrete, psi = $57{,}000\sqrt{f_c'}$

E_{cr} = long-term or creep modulus of elasticity of concrete, psi

EI = expansion index

ES = prestress loss due to elastic shortening of concrete, kip

E_{st} = modulus of elasticity of non-prestressed reinforcement, psi

e = base of natural (Naperian) logarithms

e_1, e_2 = void ratios corresponding to respective overburden pressures P_1 and P_2

e_m = edge moisture variation distance: distance measured inward from slab edge in which soil moisture content may vary, ft

e_p = eccentricity of post-tensioning force: distance between CGS and CGC; positive when CGS is above CGC and negative when CGS is below CGC, in.

F_f = fabric factor used to modify unsaturated diffusion coefficient (α) for presence of roots, layers, fractures, and joints

COMMENTARY

Unless specific testing shows a refined value is justified, E_{cr} may be assumed to be $0.5 \times E_c$.

RECOMMENDATIONS

f = applied flexural concrete stress, psi

f_{bp} = allowable bearing stress under tendon anchors, psi

f_c = allowable compressive flexural stress in concrete, psi

f_c'= specified compressive strength of concrete at 28 days, psi

f_{ci}'= concrete compressive strength at time of stressing tendons, psi

f_{cr} = concrete modulus of rupture: flexural tension stress that produces cracking, psi

f_e = effective tendon stress after losses due to elastic shortening, creep and shrinkage of concrete, and steel relaxation, psi

f_p = minimum average of effective compressive stress due to prestress

$$\frac{1000P_r}{A}, \text{psi}$$

f_{pi} = allowable tendon stress immediately after stressing, psi

f_{pj} = allowable tendon stress due to tendon jacking force, psi

f_{pu} = specified tensile strength of prestressing steel, psi

f_{py} = specified yield strength of prestressing steel, psi

f_t = allowable flexural tension stress in concrete, psi

f_y = specified yield strength of non-prestressed reinforcement, psi

fc = percentage of fine clay

g = moment of inertia factor

COMMENTARY

RECOMMENDATIONS

H = thickness of uniform thickness foundation (UTF), in.

h = total depth of rib, measured from top surface of slab to bottom of the rib, in.

I = gross moment of inertia of cross section, in.^{4}

I_m = Thornthwaite moisture index

k = depth-to-neutral-axis ratio

k_s = soil subgrade modulus, lb/in.^{3}

L = foundation length (or length of design rectangle) in direction being considered (short L_S or long L_L), perpendicular to W, ft

LL = liquid limit, %

L_L = long dimension of design rectangle, ft

L_S = short dimension of design rectangle, ft

M_L = maximum applied service load moment in long direction from either edge drop or edge lift; positive if producing tension at bottom of foundation, negative if producing tension at top of foundation, ft-k/ft

COMMENTARY

This index I_m is derived from agricultural soil science[3] and is based, on average, over an extended period of time (for example, 20 or 30 years) of the rainfall in excess or deficit of average evapotranspiration rates. An I_m of zero would indicate that, on average, rainfall equals the evapotranspiration over an extended period of time. An I_m that is negative indicates a sustained moisture deficit averaged over an extended period of time. Similarly, a positive I_m indicates moisture in excess of the evapotranspiration rate averaged over an extended period time. Maps are included in the appendix of this standard to estimate the I_m in various parts of the United States (Appendix Fig. A.1), with enlarged maps of the states of Texas and California (Appendix Fig. A.2 and A.3). This long-term average I_m is correlated only with the equilibrium suction at depth in absence of overriding factors (Fig. 9.11). It should not be used to estimate the effect of surface conditions, such as lawn irrigation, or the effects of flower beds and trees. These conditions should be addressed by other methods that are in this standard and commonly require computer modeling.

RECOMMENDATIONS

COMMENTARY

M_S = maximum applied service load moment in short direction from either edge drop or edge lift; positive if producing tension at bottom of foundation, negative if producing tension at top of foundation, ft-k/ft

n = number of ribs in cross section in direction being considered

n_T = total number of tendons in direction being considered

P = uniform unfactored line load acting along entire length of perimeter ribs, which includes weight of exterior wall and those portions of superstructure dead and live loads that frame into exterior wall, excluding any foundation concrete weight, lb/ft

PI = plasticity index, %

PL = plastic limit, %

P_e = effective prestress force in tendon after losses due to elastic shortening, creep and shrinkage of concrete, and steel relaxation, kip

$$P_e = P_i - ES - CR - SH - RE$$

P_i = prestress force in tendon immediately after stressing and anchoring tendons considering effects of tendon friction, kips

P_r = effective prestress force in concrete after losses due to tendon friction, elastic shortening, creep and shrinkage of concrete, steel relaxation, and subgrade friction, kip

$$P_r = P_e - SG$$

P_s = prestress force at jacking end immediately before anchoring tendons, kip

P_1, P_2 = overburden soil pressures corresponding to void ratios e_1 and e_2, psi

RECOMMENDATIONS

pF = soil suction value expressed as common logarithm of height of water (in cm) that suction energy can support

q_{allow} = allowable soil bearing pressure, lb/ft^2

q_u = unconfined compressive strength of soil, lb/ft^2

RE = prestress loss due to steel relaxation, kip

r_1 = area ratio

S = interior stiffening rib spacing used for moment and shear equations, ft

S_B = section modulus with respect to bottom fiber, in.^{3}

S_S = slope of suction versus volumetric water content curve

S_T = section modulus with respect to top fiber, in.^{3}

s = spacing of rib shear reinforcement, in.

SCF = stress change factor; used in determination of y_m

SF = shape factor; unitless measure of foundation irregularity

SG = reduction in compressive force on concrete cross section caused by subgrade friction, kip

SH = prestress loss due to concrete shrinkage, kip

t = slab thickness in ribbed foundation, in.

V_L = maximum shear force in long direction under-service load from either edge drop or edge lift, kip/ft

V_S = maximum shear force in short direction under service load from either edge drop or edge lift, kip/ft

COMMENTARY

Soil suction quantifies the energy level in the soil-moisture system. An imbalance of total soil suction between either the environment or adjacent soil tends to drive moisture toward a higher soil suction value. Soil suction can be expressed as *pF*, which is the logarithm to the base 10 cm of a column of water that could be theoretically supported by the energy level described, as a direct measurement of the height of a column of water (in cm), or as a negative pressure in lb/ft^2. $pF = \log(\text{MPa} \times 10{,}197)$, where pF is the log of the height of an equivalent column of water (in cm) having the reference pressure at its base.

RECOMMENDATIONS

v = applied shear stress under service load, psi

v_c = allowable shear stress in concrete, psi

W = foundation width (or width of design rectangle) in direction being considered (short or long), perpendicular to L, ft

W_{slab} = foundation weight, lb

w = unit weight of concrete, lb/ft^3

y_m = maximum unrestrained differential soil movement, in.

$y_{m\ shrink}$ = y_m value for edge drop, in.

$y_{m\ swell}$ = y_m value for edge lift, in.

z = smaller of L or 6β in direction considered, ft

z_m = moisture active zone: depth below soil surface at which suction varies by less than 0.027 pF/ft

α = unsaturated diffusion coefficient: measure of moisture movement in unsaturated soils

α′ = unsaturated diffusion coefficient modified by soil fabric factor: $\alpha' = \alpha F_f$

COMMENTARY

If the soil beneath the slab experiences a change in its moisture content after construction of the slab, it will distort into either an edge drop mode (also termed "edge drying," "center heave," "center lift," or "doming") or an edge lift mode (also called "edge swell," "edge heave," or "dishing").

The amount of differential soil movement y_m to be expected depends on a number of conditions, including the type and amount of clay mineral, depth of clay layers, uniformity of clay layers, the initial wetness, the depth of the active zone (depth of soil suction variation), and the velocity of moisture infiltration or evaporation, as well as other less easily measured and controlled effects.

The moisture active zone z_m for expansive soils refers to the depth below the ground surface at which a change in moisture content (and hence a change in suction value) can be expected due to environmental or other causes. The depth of this zone is also the location of the equilibrium moisture content, whether related to generally uniform soil conditions with environmental influences or to other conditions, such as a cemented layer or water table. The movement active zone is usually less deep than the moisture active zone due to overburden restraint.

RECOMMENDATIONS | **COMMENTARY**

α'_{shrink} = α' value for edge drop

α'_{swell} = α' value for edge lift

β = approximate distance from edge of foundation to point of maximum moment; function of relative stiffness of soil and foundation, ft

$$\beta = \frac{1}{12}\sqrt[4]{\frac{E_{cr}I}{1000}}$$

The maximum moment does not occur at the point of actual soil-slab separation but at some distance farther toward the interior. The location of the maximum moment can be closely estimated by β—a length that depends on the relative stiffness of the soil and the stiffened slab. The location of the maximum shear is between the edge of the slab and β.

γ_h = change of soil volume for unit change in suction corrected for actual percentage of fine clay; also referred to as matrix suction compression index

This is a soil property that can be determined by various means of testing, which are described in more detail later in this standard. It is analogous to the compression index used in settlement analysis in saturated soil mechanics. It is defined as the change in volume related to a change in suction for an intact specimen of soil. The change of suction is similar to the change in effective stress in settlement analysis but has a more complex relationship.

$\gamma_{h\ mod}$ = γ_h weighted for layered soils

$\gamma_{h\ mod\ shrink}$ = $\gamma_{h\ mod}$ value for center lift

$\gamma_{h\ mod\ swell}$ = $\gamma_{h\ mod}$ value for edge lift

γ_0 = change of soil volume for unit change in suction for 100% fine clay

μ = coefficient of friction between foundation and subgrade

4.0 — STRUCTURAL ANALYSIS AND DESIGN

4.1 — General

4.1.1 — Overlapping rectangles

Design criteria specified in this standard are based on a rectangular ribbed foundation. Foundation shapes that do not consist of a single rectangle shall be modeled with overlapping design rectangles that are as large as possible, with each design rectangle analyzed separately. Each design rectangle shall

C4.1.1 — Overlapping rectangles

Primary attention should be given to rectangles that most reasonably represent the main portion of the foundation. Long, narrow rectangles may not represent the overall foundation and in most cases should not govern the design. PTI DC10.1-08[2] provides examples of the overlapping rectangle method.

RECOMMENDATIONS

have slab and rib geometry consistent with that of the actual foundation within the area of the design rectangle.

Where a Secondary Design Rectangle is selected, design requirements in the short direction do not apply to the area which overlaps the Primary Design Rectangle and the Primary Design Rectangle shall control the design.

COMMENTARY

The shape factor (SF) is determined by dividing the contiguous slab perimeter dimension, squared, by the area of the contiguous slab.

SF = (foundation perimeter, ft)^2/(foundation area, ft^2).

The simplified shape factor (SSF) is determined by dividing the perimeter of the simplified shape of the combined overlapping rectangles, squared, divided by the area of the simplified shape of the combined overlapping rectangles.

SSF = (combined overlapping rectangle perimeter, ft)^2/(area of overlapping rectangles, ft^2).

Additional consideration regarding the foundation design is required when the SF is greater than 32 or the SSF is greater than 24.

The shape factor (SF) is a unitless measure of a foundation's irregularity. Experience has shown that the shape of a foundation affects its performance. For example, on the same expansive soil experiencing the same moisture changes, a small square foundation will perform differently than a large, irregularly shaped foundation.

The SF and SSF identifies those foundations, where the foundation shape necessitates additional attention in the design.

If SF exceeds 32 or the SSF exceeds 24, the designer should consider one or more of the following:

- Modifications to the foundation footprint to reduce the shape factor
- Strengthened foundation systems (additional stiffening ribs or deepened ribs in areas of high torsion or non-prestressed reinforcement)
- Geotechnical approaches (such as moisture barriers, moisture conditioning, or moisture injection) to reduce the shrink/swell potential of the supporting soils. Geotechnical approaches should reduce $y_{m\text{-}center}$ to less than 2.0 in. (5.08 cm) and $y_{m\text{-}edge}$ to less than 1.0 in. (2.54 cm).

4.1.2 — Perimeter load

When *P* varies, use the largest value for the edge drop design and the smallest value for the edge lift design.

C4.1.2 — Perimeter load

The mathematical analysis forming the basis for the equations for internal forces and deflections[4] in this standard consider perimeter loads between 600 and 1500 lb/ft.

RECOMMENDATIONS

4.1.3 — Concentrated loads

Concentrated loads shall be evaluated on an individual basis. If the slab stresses produced by concentrated loads exceed those permissible, the loads shall be framed to adjacent ribs in ribbed foundations, or a footing shall be placed below them.

4.1.4 — Loss of prestress

Effective prestress force in the concrete after all losses shall be

$$P_r = P_i - ES - CR - SH - RE - SG$$

For determination of the minimum effective prestress force *Pr*, *SG* shall be calculated as follows:

$$SG = \left(\frac{W_{slab}}{2000}\right)\mu$$

COMMENTARY

Based on successful experience with foundations built with perimeter loads up to and exceeding 2500 lb/ft that have been designed using these equations, the PTI Slab-on-Ground Committee is confident that the equations will yield reasonable results for perimeter loads in excess of those used in the research. Note that the definition of *P* includes the dead and live load in both swell modes. Removing the live load in the edge lift swell mode may result in unnecessarily conservative edge lift moments because the equations in this standard were derived from foundation-deformation computations that considered the foundation loaded with both a dead and live load. In the edge lift swell mode, designers may use the dead load and sustained live load, or dead load only, if either is judged to be appropriate.

In addition to the variable edge load *P*, internal forces and stiffness requirements specified in this standard are based on uniform applied loads acting on an entire foundation plan area of a 40 lb/ft^2 live load and a 65 lb/ft^2 dead load, representing the weight of an assumed 4 in. (10.16 cm) slab plus 15 lb/ft^2 for non-bearing partitions and other interior dead loads.

C4.1.3 — Concentrated loads

Equations for flexural stresses from concentrated loads may be derived from the beam-on-elastic foundation theory.

C4.1.4 — Loss of prestress

The effective prestressing force in post-tensioned foundations is further reduced by the frictional resistance to movement of the foundation on the subgrade during stressing, as well as the frictional resistance to dimensional changes due to concrete shrinkage, creep, and temperature variations. The largest amount of prestress loss due to subgrade friction occurs in the center of the foundation. The greatest structural requirement for prestress force, however, is at the location of the maximum moment, which occurs at approximately one β-length inward from the edge of the foundation.

ES, *CR*, *SH*, and *RE* can be calculated with generally accepted methods for estimating losses in prestressed concrete.[5] Total prestress loss (after the effects of tendon friction) is the sum of *ES*, *CR*, *SH*, and *RE*. In lieu of calculating such losses, a value of $P_e = (0.7f_{pu} - 15 \text{ ksi}) \times A_{ps}$ may be assumed for the low-relaxation strand.

RECOMMENDATIONS

For determination of the effective prestress force P_r used in the flexural and shear stress calculations, SG shall be calculated as follows

$$SG = \left(\frac{W_{slab}}{2000}\right)\left(\frac{\beta}{L/2}\right)\mu$$

where β and L are in the direction being considered.

COMMENTARY

The expression for P_i assumes a high-side friction "wobble" coefficient of 0.002 (refer to ACI 423.10R-16, Table 4.4.2),[6] and one-end tendon stressing (that is, P_i is assumed to act at the far end of the tendon). In lieu of more detailed analysis, a value of $P_i = P_s/(1 + 0.002L)$ may be used. Typically, $P_s = 0.8A_{ps}f_{pu}$.

SG does not directly affect the tendon force. However, it has the same effect as reducing the prestress force acting on the concrete cross section and, therefore, for simplicity, can be conveniently and mathematically grouped with the other factors that actually affect the force in the tendon. The expression for SG used for the determination of the minimum average compressive stress due to prestress represents the maximum effect of subgrade friction, which occurs at the center of the foundation, where the frictional force-resisting movement is based on the weight of half of the slab—that is, $W_{slab}/2$. Because the maximum structural requirement occurs at a distance β from the edge of the foundation, the expression for SG used to determine the flexural and shear stress equations represents the prestress force at the location of the maximum structural requirement.

An extensive review of the technical literature was made in order to determine the value of the coefficient of friction that might be expected during tendon stressing. As a result of this review three factors were identified as having an important effect upon the coefficient of friction. These factors are: 1) the amount of movement the slab experiences as a result of shrinkage and temperature effects between the time it is cast and the time it is prestressed, 2) temperature of soil at time of stressing, and 3) the material over which sliding occurs.

A large force is required to induce movement when the slab has not been previously moved. Once this "first movement" displacement has occurred, subsequent movements require only a fraction of the force initially necessary for movement. Research also shows that if slab movements remain very small, the coefficient is also smaller than the maximum value.

Figure C4.1 is representative of the effect different sliding mediums have on the magnitude of the friction coefficient. As can be seen in Fig. C4.1, even if polyethylene is not required or specified as a vapor retarder, it may prove desirable to place it below the slab in order to achieve a reduction in the friction coefficient.

RECOMMENDATIONS

4.2 — Ribbed foundations

Calculations for ribbed foundations shall be based on criteria specified in Sections 4.2.1 to 4.2.4.

COMMENTARY

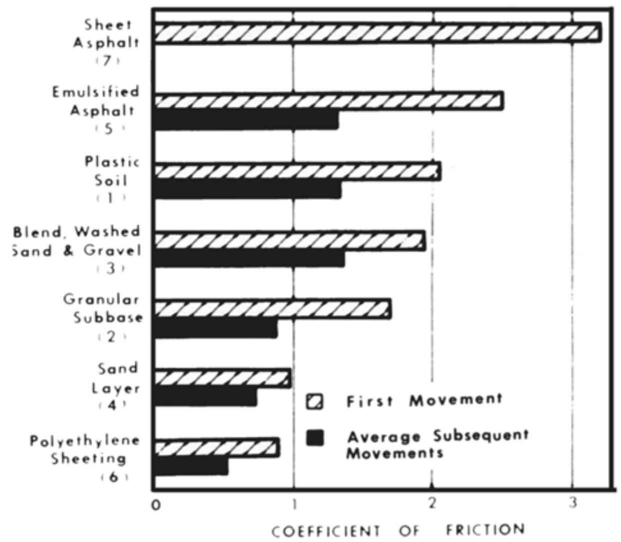

Fig. C4.1 Summary of Coefficients of friction for 5 in. slabs

Measured slab movements indicate that concrete placement during hot weather results in effective coefficient of friction values in the range of 0.50 - 0.60 for uniform thickness foundations cast on polyethylene sheeting.

Concrete placement during cold weather may result in higher coefficients. The effective coefficient for these conditions ranges between 0.60 and 0.75 for polyethylene.

For slabs cast directly on a sand layer, the coefficient has an effective value between 0.75 and 1.00.

Ribs act to increase friction. Thus, μ values of 0.75 and 1.00 for polyethylene sheeting and sand, respectively, appear to be reasonable design values for ribbed foundations.

For normal construction practices, μ should be taken as 0.75 for slabs on polyethylene and 1.0 for slabs cast directly on a sand base. For other materials, refer to Fig. C4.1.

C4.2 — Ribbed foundations

Equations in this standard for internal forces and stiffness requirements are based on shallow ribbed foundations. Ribbed foundation variables appearing in these equations are L, S, h, P, e_m, and y_m, as defined in Section 3.0. Limitations and constraints for these variables are stated in this section. The equations are valid for ribbed foundations that are in conformance with these limitations.

RECOMMENDATIONS

4.2.1 — Minimum slab thickness

Minimum slab thickness t shall be 4 in. (100 mm).

4.2.2 — Ribs

4.2.2.1 — Minimum size

4.2.2.1.1 — Rib depth

Minimum rib depth h shall be the larger of (t + 7) in. ([t + 180] mm) or 11 in. (280 mm). When more than one rib depth is used in the calculations, the ratio between the maximum and minimum rib depths shall not exceed 1.2.

4.2.2.1.2 — Rib width

Rib width used in section property calculations shall neither be less than 6 in. (150 mm) nor greater than 14 in. (360 mm).

COMMENTARY

Conditions exist that require larger gross section properties than required to resist the applied forces due to swelling clays. Geometry resulting in larger gross section properties may be used for actual construction. For example, frost depth often requires the use of perimeter ribs that are substantially deeper than those required in the design for expansive soil movement. Designers should consider the use of additional reinforcement in these deeper rib sections.

C4.2.2.1.1 — Rib depth

The depth of ribs h is usually the controlling parameter in the structural design of ribbed foundations. Rib depth is the structural parameter that most influences the moment capacity and shear capacity in the ribbed foundation. The equations for internal forces and stiffness in this standard were derived assuming a uniform moment of inertia across the full width of the foundation, implying that all ribs are the same depth.[4] Successful experience exists, however, supporting the use of different rib depths in design (such as a deeper edge rib), provided that the depths do not vary by more than 20%.

C4.2.2.1.2 — Rib width

The width of ribs b affects the soil-bearing capacity, the applied shear stress, and all section properties. To ensure the accuracy of equations for applied service moments, shears, and stiffness (in which b does not appear), the rib width used in section property calculations must be limited to a range of 6 to 14 in. (150 to 360 mm). Within this range, the flexural design is virtually unaffected by the rib width. Based on successful experience, it is permissible to use ribs of different widths. Nonformed ribs less than 8 in. (200 mm) wide may be impractical due to excavation considerations. Rib widths greater than 14 in. (360 mm) may be used if required for bearing. In that case, however, a width of 14 in. (360 mm) shall be used in section property calculations. Excavated rib widths most commonly found in practice are 10 to 12 in. (250 to 305 mm).

RECOMMENDATIONS

4.2.2.2 — Rib spacing

Rib spacing S used in actual construction shall be a maximum of 15 ft (4.6 m). S used in moment and shear equations shall be the average rib spacing if the ratio between the largest and the smallest spacing does not exceed 1.5. If the ratio between the largest and the smallest spacing exceeds 1.5, S used in moment and shear equations shall be 0.85 times the largest spacing. S used in moment and shear equations shall neither be less than 6 ft (1.8 m) nor greater than 15 ft (4.6 m). The rib spacing used in the section properties shall be the actual rib spacing.

4.2.2.3 — Rib continuity

Ribs used in design calculations shall be continuous between the edges of the foundation in both directions.

4.2.3 — Minimum prestress force for ribbed foundations

The effective prestress force P_r shall not be less than $0.05A$ (kip). P_r shall be determined using the prestress at mid-slab or the location of the minimum prestress.

4.2.4 — Soil-bearing pressure

Applied soil-bearing pressure shall be evaluated using generally accepted techniques and shall not exceed q_{allow} as specified by the LDP with geotechnical experience.

4.3 — Uniform thickness foundations (UTFs)

Any ribbed foundation conforming to all requirements of this standard (except Sections 4.2.4 and 5.4) are permitted to be converted to an equivalent UTF, as specified herein. Converted UTFs must satisfy all requirements of Sections 5.0, 6.0, and 7.0.

COMMENTARY

C4.2.2.2 – Rib spacing

For ribbed foundations, the location of ribs is dictated mainly by the configuration of the foundation system, the structural design requirements, and the wall layout of the superstructure.

Additional ribs may be required where heavy loads are applied to the foundation, as in the case of a fireplace or an interior column.

C4.2.2.3 — Rib continuity

The design method is based on full continuity of ribs from edge to edge of the foundation in both directions. Ribs should extend across both full plan dimensions whenever possible. When architectural considerations (openings, corners, irregularities in plan shape, and so on) prevent rib continuity, the designer must provide equivalent rib continuity using rational engineering approaches.

To be considered as a continuous rib in the design rectangle, the rib should:

(a) Overlap a parallel rib with adequate length; or

(b) Be connected to a parallel rib by a perpendicular rib, which transfers by torsion the bending moment in the rib.

C4.2.3 — Minimum prestress force for ribbed foundations

If excessive shrinkage cracking is anticipated, the designer should consider increasing the minimum force to $0.1A$(kip) and details to minimize restraint to shortening.

C4.2.4 — Soil-bearing pressure

Refer to PTI DC10.1-08[2] for one method of determining the applied soil-bearing pressure. Other generally accepted techniques may be used.

C4.3 — Uniform thickness foundations (UTFs)

When converting a ribbed foundation to a UTF, the ribbed foundation must satisfy all requirements applicable to ribbed foundations, with the exception of soil bearing (refer to Section 4.2.4) and cracked section provisions (refer to Section 5.4). The converted UTF must conform to the flexural stress criteria in Section 5.0 (including the

RECOMMENDATIONS

COMMENTARY

cracked section requirements in Section 5.4), shear criteria in Section 6.0, and minimum stiffness requirements in Section 7.0. (Note that β distances can be different in the conformant ribbed foundation and the converted UTF.)

4.3.1 — UTF conversion

Minimum thickness shall be

$$H = \sqrt[3]{\frac{I}{W}}$$

where H is in in.; I is in in.^{4}; and W is in ft.

H shall be calculated for each direction (long and short) and the maximum value shall be used. H shall not be less than 7.5 in. (190 mm) unless a continuous rib, conforming to Section 4.2.2.1, is provided along the entire perimeter.

C4.3.1 — UTF conversion

The conversion from ribbed foundation to UTF is based on equal moments of inertia. Units of the uniform thickness conversion equation are not immediately obvious. The equation is derived as follows:

The gross moment of inertia I for a rectangular UTF is

$$I = \frac{(12W)H^3}{12}$$

where H is in in.; I is in in.^{4}; and W is in ft.

4.3.2 — Minimum prestress force for UTFs

The effective prestress force P_r shall not be less than 0.05A (kip). P_r shall be determined using the prestress at mid-slab or the location of the minimum prestress.

C4.3.2 — Minimum prestress force for UTFs

The required minimum force per unit of the cross-sectional area in the UTF is the same as that for the ribbed foundation (Section 4.2.3). This results in substantially larger total prestress force in the UTF than in the equivalent ribbed foundation because the cross-sectional area of the UTF is always larger than that of the ribbed foundation.

4.3.3 — Soil-bearing pressure

Applied soil-bearing pressure shall be evaluated using generally accepted techniques and shall not exceed q_{allow} as specified by the LDP with geotechnical experience.

C4.3.3 — Soil-bearing pressure

Refer to PTI DC10.1-08[2] for one method of determining the applied soil-bearing pressure. Other generally accepted techniques may be used.

5.0 — FLEXURE

Concrete flexural stresses shall be calculated as follows

$$f = \frac{1000P_r}{A} \pm \frac{12{,}000M_{L,S}}{S_{T,B}} \pm \frac{1000P_r e_p}{S_{T,B}}$$

C5.0 — FLEXURE

The sign convention used in this standard considers concrete tension stresses to be negative and compression stresses positive. Therefore, the absolute values should be used when comparing to allowable stresses.

The maximum moment will vary depending on the swelling mode and the direction being designed. Wray[4] provides background and derivations of the equations specified in Section 5.0.

RECOMMENDATIONS

Maximum moment M shall be as specified in Sections 5.1 and 5.2. P_r shall be calculated at the point of maximum moment, which is at distance β from the edge of the slab.

5.1 — Edge drop

5.1.1 — Long direction

$$M1 = A_o\left[B(e_m)^{1.238} + C\right]$$

where

$$A_o = \frac{1}{727}\left[(L)^{0.013}(S)^{0.306}(h)^{0.688}(P)^{0.534}(y_m)^{0.193}\right]$$

and for $0 \le e_m \le 5$

$B = 1$ and $C = 0$.

and for $e_m > 5$

$$B = \left(\frac{y_m - 1}{3}\right) \le 1.0$$

$$C = \left[8 - \frac{P - 613}{255}\right]\left[\frac{4 - y_m}{3}\right] \ge 0$$

5.1.1.a — Compute $M1$ at e_m from geotechnical report

5.1.1.b — Compute 5 ft (1.5 m) threshold: $M1$ at e_m = 5 ft (1.5 m) using equation in 5.1.1.

5.1.1.c — The moment to be used for design is the larger value in magnitude between that computed in 5.1.1.a or 5.1.1.b given by the expression:

$$M1 = max(M1_{e_m}, M1_{5ft})$$

COMMENTARY

C5.1 — Edge drop

C5.1.1 — Long direction

Licensed design professionals should ensure that calculations of edge drop moments based on values of e_m greater than 5 ft (1.5 m) should not be less than those generated at the 5 ft (1.5 m) threshold. There is a discontinuity in the equations in the long direction edge drop moments at e_m = 5 ft (1.5 m) (Eq. (7.1.1)) The moment for e_m slightly greater than 5 ft (1.5 m) is often less than the moment with e_m exactly equal to 5 ft (1.5 m).

RECOMMENDATIONS

COMMENTARY

5.1.2 — Short direction

$$M2 = \left(\frac{58 + e_m}{60}\right) M1$$

5.1.3 — Design Moments

For L_L < 75 ft:

For $L_L/L_S > 1.15$: $M_L = M1$ and $M_S = M2$

For $L_L/L_S <= 1.15$ and $L_L/L_S > 1.1$:

For $L_L/L_S <= 1.1$: $M_L = M1$ and $MS = M1$

For L_L > 75 ft:

$M_L = (M1 + M2) / 2$
$M_S = (M1 + M2) / 2$

5.2 — Edge lift

5.2.1 — Long direction

$$M1 = \frac{S^{0.1}(he_m)^{0.78}(y_m)^{0.66}}{7.2L^{0.0065}P^{0.04}}$$

5.2.2 —Short direction

$$M2 = h^{0.35}\left[\frac{19 + e_m}{57.75}\right] M1$$

5.2.3 — Design Moments

For L_L < 75 ft:

For $L_L/L_S > 1.15$: $M_L = M1$ and $M_S = M2$

For $L_L/L_S <= 1.15$ and $L_L/L_S > 1.1$:

$M_L = M1$
$M_S = (M1 + M2) / 2$

For $L_L/L_S <= 1.1$: $M_L = M1$ and $M_S = M1$

RECOMMENDATIONS

For L_L > 75 ft:

$ML = (M1 + M2) / 2$

$MS = (M1 + M2) / 2$

5.3 — Allowable stress

Concrete flexural stress calculated in accordance with Section 5.0 shall not exceed the following

Tension: $f_t = 6\sqrt{f'_c}$

Compression: $f_c = 0.45f'_c$

5.4 — Cracked sections

Sufficient reinforcement prestressed or nonprestressed in any combination shall be provided to develop $0.5M_L$ and $0.5M_S$ for both swell modes, using conventional cracked-section flexural strength methods.

5.4.1 — Tensile force in prestressed reinforcement shall be taken as P_e and tensile force in nonprestressed reinforcement shall be taken as $A_s f_y/2$.

5.4.2 — Nonprestressed reinforcement, if required, shall be placed perpendicular to the perimeter of the foundation, starting with minimum concrete cover from the foundation edge and extending inward with a minimum length of 2β.

6.0 — SHEAR

Applied concrete shear stress v produced by V_L or V_S shall be calculated as follows:

6.1 — Applied concrete shear stress

6.1.1 — Ribbed foundations

$$v = \frac{1000(V_L \text{ or } V_S)}{nbh}$$

COMMENTARY

C5.3 — Allowable stress

The sign convention used in these equations considers concrete tension stresses to be negative and compression stresses positive. Therefore, the absolute values should be used when comparing them to allowable stresses.

C5.4 — Cracked sections

Because of the post-cracking increase in soil support adjacent to the crack, equivalency does not require reinforcement for the full values of M_L and M_S. After considerable study, it was decided that reasonable equivalency is provided throughout a wide range of soil and foundation parameters by providing reinforcement for $0.5M_L$ and $0.5M_S$. Bondy[7] addresses types of cracking and their ramifications in post-tensioned residential foundations.

C6.0 — SHEAR

The area resisting applied shear is based on the web area of the ribs alone, consistent with generally accepted structural engineering practice. Wray[4] provides background and derivations of the equations specified in Section 6.0.

RECOMMENDATIONS

COMMENTARY

6.1.2 — UTFs

$$v = \frac{1000(V_L \text{ or } V_S)}{A}$$

Maximum shear force V shall be as specified in Sections 6.2 and 6.3.

6.2 — Edge drop

6.2.1 — Long direction

$$V_L = \frac{1}{1940}\left(L^{0.09}S^{0.71}h^{0.43}P^{0.44}{y_m}^{0.16}{e_m}^{0.93}\right)$$

For $y_m \leq 1$ in. (25 mm), e_m should not exceed 5 ft (1.5 m) for shear only.

6.2.2 — Short direction

For $L_L/L_S \geq 1.1$

$$V_L = \frac{1}{1350}\left(L^{0.19}S^{0.45}h^{0.20}P^{0.54}{y_m}^{0.04}{e_m}^{0.97}\right)$$

For $L_L/L_S < 1.1$, $V_S = V_L$

For $y_m \leq 1$ in. (25 mm), em should not exceed 5 ft (1.5 m) for shear only.

6.3 — Edge lift

6.3.1 — Long and short direction

$$V_L = V_S = \frac{L^{0.07}h^{0.4}P^{0.03}{e_m}^{0.16}{y_m}^{0.67}}{3S^{0.015}}$$

RECOMMENDATIONS

6.4 — Allowable stress

Applied shear stress v calculated in accordance with Section 6.0 shall not exceed the following

$$v_c = 2.4\sqrt{f'_c} + 0.2\left(1000\frac{P_r}{A}\right)$$

The effective prestress force P_r shall be determined using the prestress at β.

COMMENTARY

C6.4 — Allowable stress

If v exceeds v_c, provide shear reinforcement in accordance with the following

$$\frac{A_v}{S} = \frac{(v - v_c)b}{0.4f_y}$$

Possible alternatives to shear reinforcement include:

(a) Increasing the rib depth;
(b) Increasing the rib width; and
(c) Increasing the number of ribs (decrease the rib spacing).

7.0 — STIFFNESS

Foundation stiffness $E_{cr}I$ in both short and long directions and for both soil swelling modes shall conform to the following

For Edge Drop:

$$E_{cr}I_{L\ or\ S} = 5{,}760{,}000 * M_{L\ or\ S}L_{S\ or\ L}C_S Z_{L\ or\ S}$$

For Edge Lift:

$$E_{cr}I_{L\ or\ S} = 11{,}520{,}000 * M_{L\ or\ S}L_{S\ or\ L}C_S Z_{L\ or\ S}$$

C7.0 — STIFFNESS

Differential foundation deflection is controlled by providing minimum foundation stiffness in accordance with the equation presented, which is applicable to both edge lift and edge drop swell modes.

This equation was derived by relating permissible deflection and the slab length over which it occurs[2,8] to an assumed parabolic shape. This method for controlling differential deflections, which directly relates foundation stiffness to permissible curvatures and deflections, is simpler and reasonably equivalent to differential deflection criteria presented in previous editions of this standard. The minimum stiffness $E_{cr}I$ required should be determined for each direction considering both swell modes. The coefficient C_s is a function of the type of superstructure material and the swelling condition (edge drop or edge lift).

Bondy[9] discusses the relationship between construction effects and actual deflections in greater detail.

Significant problems (severe drywall cracking, large wall/ceiling separations) are evident in residential wood-framed structures with prefabricated long-span roof trusses, when the trusses are rigidly attached to nonbearing partition walls between the truss supports. In that case, even a small relative vertical movement between the two ends of the extremely rigid trusses can cause distress. To mitigate this condition, Table C7.1 requires very high C_s values (resulting in very large required stiffness values) when prefabricated roof trusses are used, regardless of the superstructure material. C_s values specified in Table C7.1 for prefabricated roof trusses may be waived, and smaller values based on the

RECOMMENDATIONS

COMMENTARY

appropriate superstructure material may be used if joinery details are specified that permit relative vertical movement between prefabricated roof trusses and intersecting nonbearing partition walls while providing required lateral bracing. Smaller values of C_s may be used for other superstructure materials listed in Table C7.1 if effective jointing details are used to minimize cracking, such as closely spaced control joints in brick or stucco walls.

Table R7.1—Recommended values of stiffness coefficient C_s

Building material	C_s
Wood and fiber cement siding	0.50
Stucco, plaster, and adhered masonry	0.75
Anchored masonry (stone and brick)	1.00
Concrete masonry units	2.00
Prefabricated roof trusses (without steps to minimize truss lift)*	2.08

*Trusses that span across full length or width of foundation from edge to edge.

8.0 — GENERAL

Internal forces and stiffness requirements specified in this standard are based on criteria in this section.

C8.0 — GENERAL

8.1 — Soils

This standard is applicable to foundations built on expansive soils, as defined in Section 8.1.2.

C8.1 — Soils

8.1.1 — Field investigation and laboratory testing

The minimum field investigation and laboratory testing program shall be determined by a licensed design professional (LDP) based on local practice and experience.

8.1.2 — Expansive soils

Soils must satisfy each of Sections 8.1.2.1 through 8.1.2.3 or satisfy Section 8.1.2.4 to be considered expansive.

8.1.2.1 — Plasticity index (*PI*) is 15 or greater, determined in accordance with ASTM D4318 and a weighting procedure using three 5 ft (1.5 m) layers with a weight of 3 for the top layer, 2 for the middle layer, and 1 for the bottom layer; or using the *PI* of a 2 ft (0.60 m) or thicker layer within the upper 5 ft (1.5 m) with a *PI* of 15 or greater.

C8.1.2 — Expansive soils

This definition of expansive soils is consistent with soil classification criteria presented in the International Building Code (IBC).

RECOMMENDATIONS

8.1.2.2 — More than 10% of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D422 and a weighting procedure using three 5 ft (1.5 m) layers determined using the depth weighting procedures of Section 8.1.2.1, disregarding the 2 ft (0.60 m) or thicker layer provisions.

8.1.2.3 — More than 10% of the soil particles are less than 5 μm in size, determined in accordance with ASTM D422 and a weighting procedure using three 5 ft (1.5 m) layers determined using the depth weighting procedures of Section 8.1.2.1, disregarding the 2 ft (0.60 m) or thicker layer provisions.

8.1.2.4 — Expansion index (EI) is greater than 20, determined in accordance with ASTM D4829 and a weighting procedure using three 5 ft (1.5 m) layers determined using the depth weighting procedures of Section 8.1.2.1, disregarding the 2 ft (0.60 m) or thicker layer provisions.

9.0 — SOIL PARAMETERS

e_m and y_m shall be determined by the procedures in Sections 9.1 and 9.2 or Section 9.3.

COMMENTARY

C9.0 — SOIL PARAMETERS

This standard should not be used in conjunction with any previous manual editions or standards issued by PTI.

If e_m and y_m were calculated using previous editions or standards, then the foundation must be designed using the structural procedures prescribed in corresponding previous editions or standards.

The procedure described in Sections 9.1 and 9.2 for the determination of soil support parameters for shallow foundations on expansive clay soil sites uses a rational means for evaluating the edge moisture variation distance e_m and the differential soil movement y_m. This procedure provides the ability to model soil conditions by incorporating extensive databases and research from the USDA Natural Resources Conservation Service National Soil Survey Center,[10] and by allowing for more flexibility in evaluating vertical moisture barriers, planter areas, and variable soil suction values controlling the suction conditions at the surface of the soil profile.

RECOMMENDATIONS

COMMENTARY

9.1 — Edge moisture variation distance e_m

C9.1 — Edge moisture variation distance e_m

The edge moisture variation distance is the distance beneath the edge of a shallow foundation within which moisture will change due to wetting or drying influences around the perimeter of the foundation.

The major factor in determining the edge moisture variation distance is the unsaturated diffusion coefficient α. This, in turn, depends on suction, permeability, and cracks in the soil. With the same diffusion coefficient, the e_m value will be larger for the edge drop case in which moisture is withdrawn from soil around the perimeter of the foundation. The e_m value will be smaller for an edge lift case in which moisture is drawn beneath the perimeter of the building into drier soil. Roots, layers, fractures, or joints in a CH soil (refer to Table 9.1) will increase the diffusion coefficient and increase the e_m value for both edge lift and edge drop conditions.

Calculating e_m involves the use of the Thornthwaite moisture index I_m approach and an in-place soil-based approach, which are compared using estimates based on the in-place unsaturated diffusion coefficient calculated from simple soil properties.

If the area developed is changed from a natural condition to support man-made improvements and landscaping, these anticipated changes should be incorporated into this analysis.

9.1.1 — Soil parameters

For each distinct soil layer to a depth of z_m, determine the following soil parameters:

9.1.1.1 — *LL* is liquid limit determined in accordance with ASTM D4318, %

9.1.1.2 — *PL* is plastic limit determined in accordance with ASTM D4318, %

9.1.1.3 — *PI* is plasticity index determined in accordance with ASTM D4318, %

9.1.1.4 — Percentage of soil passing No. 200 sieve = $\%_{-200}$

9.1.1.5 — Percentage of soil finer than 2 μm = $\%_{-2\mu}$, expressed as a percentage of the total sample

C9.1.1 — Soil parameters

Depths greater than 9 ft (2.7 m) may be used if justified by geotechnical analysis.

RECOMMENDATIONS

9.1.1.6 — Percentage of fine clay

$$\%fc = \left(\frac{\%_{-2\mu}}{\%_{-200}}\right)100$$

9.1.2 — Matrix suction compression index γ_h

For each significant soil layer described in Section 9.1.1, determine γ_h for swelling and shrinkage in accordance with one of the following methods:

9.1.2.1 — Method one: mineral classification and zone chart method

9.1.2.1.1 — Determine mineral classification zone (I through VI) from Fig. 9.1.

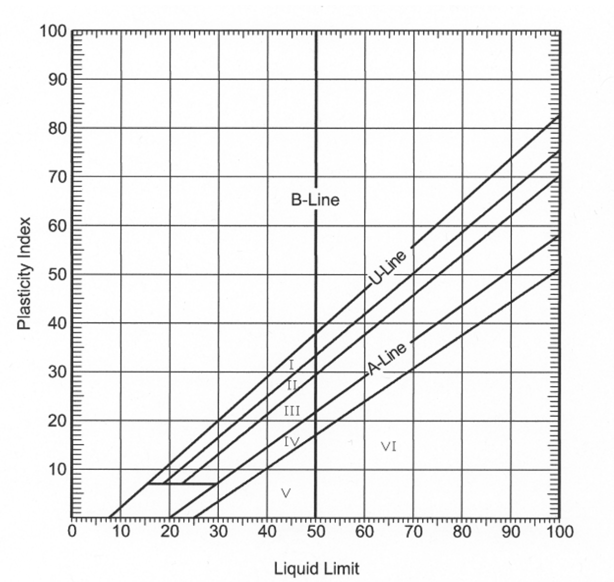

Fig. 9.1—Mineral classification chart.

9.1.2.1.2 — Determine γ_o from Fig. 9.2 to 9.7.

COMMENTARY

C9.1.2.1.1 — If data does not fall within one of the six zones, use the nearest zone. No data should plot above the U-line. If data plots within the area below a PI of 7, bounded by the U-line and the A-line, use $\gamma_0 = 0.01$.

C9.1.2.1.2 — Interpolate between γ_0 lines. Beyond extreme contour values, use the nearest values for γ_0. Figures 9.2 through 9.7 were derived from the National Soil Survey Center, USDA.[10]

RECOMMENDATIONS

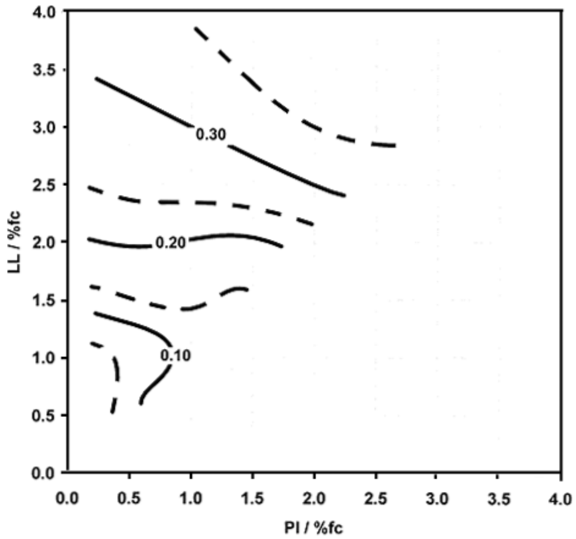

Fig. 9.2—Zone I chart for determining γ_0.

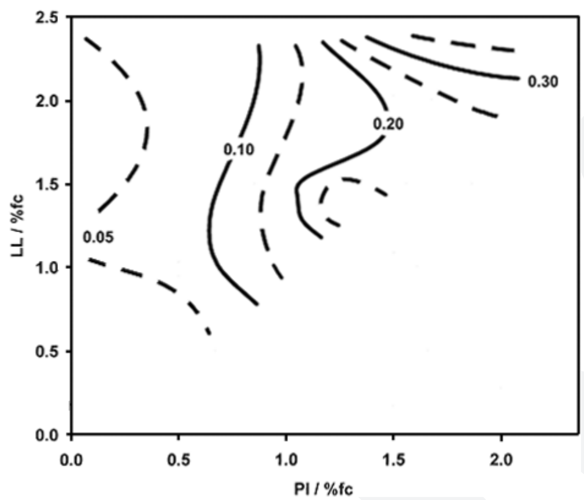

Fig. 9.3—Zone II chart for determining γ_0.

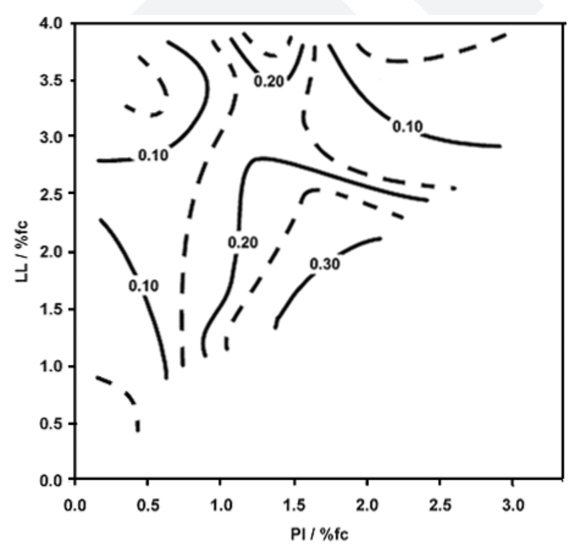

Fig. 9.4—Zone III chart for determining γ_0.

COMMENTARY

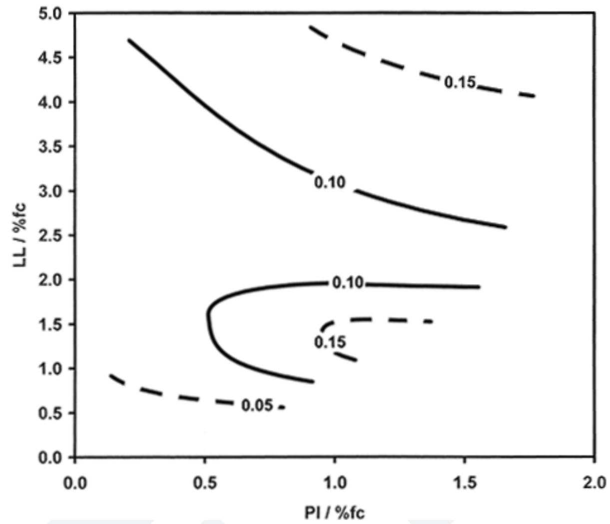

Fig. 9.5—Zone IV chart for determining γ_0.

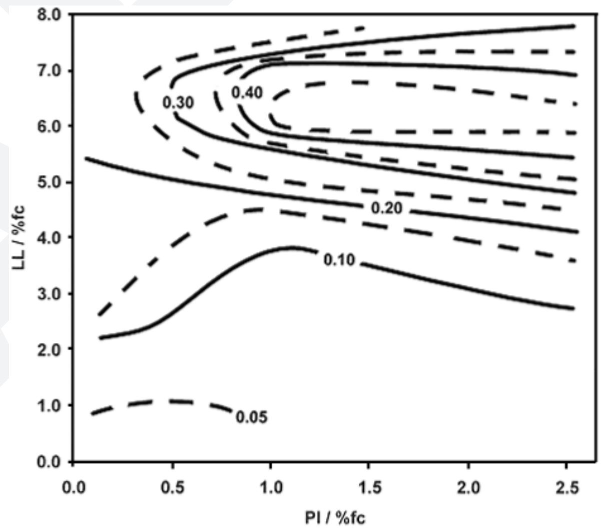

Fig. 9.6—Zone V chart for determining γ_0.

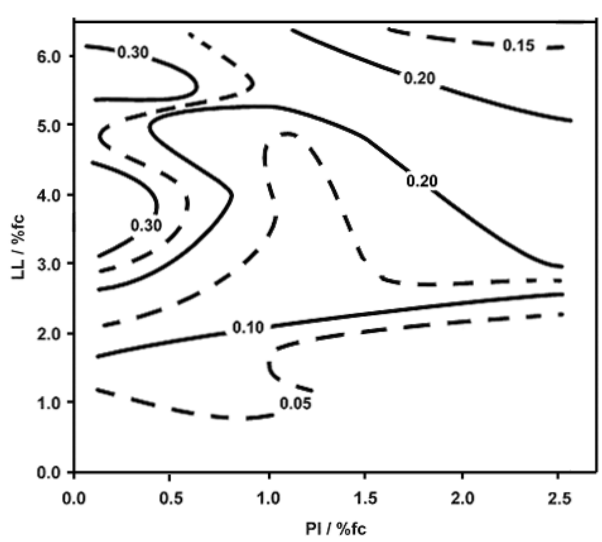

Fig. 9.7—Zone VI chart for determining γ_0.

RECOMMENDATIONS

9.1.2.1.3 — Correct γ_0 for the actual percentage of fine clays

$$\gamma_h = \frac{\gamma_0 \%fc}{100}$$

9.1.2.1.4 — Correct γ_h for swelling or shrinkage:

For swelling (edge lift): $\gamma_{h\ swell} = \gamma_h e^{\gamma h}$

For shrinkage (edge drop): $\gamma_{h\ swell} = \gamma_h e^{-\gamma h}$

9.1.2.1.5 — Correction of γ_h for coarse-grained soil. The correction of γ_h for coarse-grained soil shall only be used in cases where the percentage retained on the No. 10 sieve is 10% or more.

$$(\gamma_h)_{corr} = \gamma_h \left[\frac{100}{F\left(\frac{\gamma_{moist}}{\gamma_{in-situ}}\right) + (100 - F)} \right]$$

$$F = \frac{100}{1 + \left(\frac{J}{100 - J}\right)\left(\frac{\gamma_{moist}}{\gamma_w (G_S)_{coarse}}\right)}$$

where F is percent by volume of the fraction of the soil smaller than the No. 10 sieve (2.0 mm [0.08 in.]) as a percentage of the total soil volume; γ_{moist} is the total unit weight of the soil at the soil wet limit around a *pF* of 2.5 for clay; $\gamma_{in\text{-}situ}$ is the dry unit weight of the soil at its natural water content (around standard proctor optimum water content or shrinkage limit); J is the percent of the soil by weight that is larger than the No. 10 sieve (2.0 mm [0.08 in.]); $(G_s)_{coarse}$ is the specific gravity of the soil particles larger than 2.0 mm (0.08 in.); and γ_w is the unit weight of water.

COMMENTARY

C9.1.2.1.5 — Correction of γ_h for coarse-grained soil. The formula for γ_h is predicated on all of the soils being finer than the No. 200 sieve. Many expansive soils have substantial portions that are larger than this and the chart value of γ_h must be corrected for the percent of the soil that is larger than the No. 200 sieve. The correction must be done on a volumetric rather than weight basis. The correction method recommended herein is adapted from the method that was developed by the U.S. Department of Agriculture Natural Resources Conservation Service (NRCS).[11]

This volumetric correction will reduce the γ_h value for all soil particles larger than the No. 10 sieve (2.0 mm [0.08 in.]). The NRCS11 found that no reduction in the γ_h value is warranted for soils with particles smaller than the No. 10 sieve.

The values of γ_{moist} and $\gamma_{in\text{-}situ}$ should be for the soil in its natural state and may be estimated for the purpose of this correction.

In lieu of specific laboratory testing, $(G_s)_{coarse}$ may be assumed to be 2.65.

RECOMMENDATIONS

9.1.2.2 — Method two: expansion index (*EI*) procedure

Use ASTM D4829 to determine *EI*

$$\gamma_{h\ swell} = \frac{EI}{1700}\text{, and}$$

$$EI = 1000 \times (\text{final thickness} - \text{initial thickness})/(\text{initial thickness})$$

9.1.2.3 — Method three: consolidation-swell pressure test procedure

Use ASTM D4546, Method C

$$\gamma_{h\ swell} = \frac{(0.7)(C_S)}{(1+e_2)}$$

$$C_S = \frac{(e_1 - e_2)}{\log(P_2) - \log(P_1)}$$

Figure 9.8 shows the void ratio versus the overburden pressure.

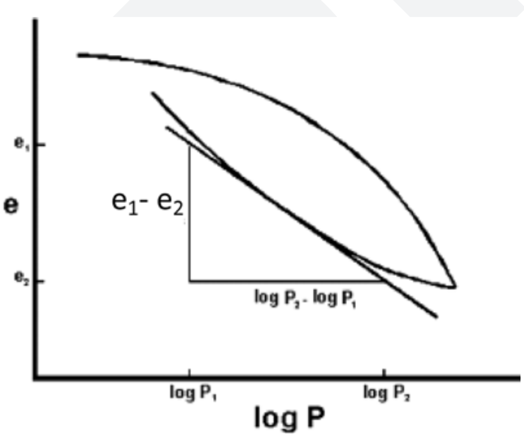

Fig. 9.8—Void ratio versus overburden pressure.

COMMENTARY

C9.1.2.2 — Method two: expansion index (*EI*) procedure

The *EI* procedure uses a remolded specimen and requires a laboratory effort approximately equivalent to the procedure that was discussed previously using the hydrometer and Atterberg limits.

C9.1.2.3 — Method three: consolidation-swell pressure test procedure

The consolidation-swell pressure test is a lengthy and expensive test, but the results are reasonably reliable.

RECOMMENDATIONS

C_s is the slope of the rebound limb of the e-log$P$ plot. $C_s = (e_1 - e_2)/(\log P_2 - \log P_1)$, where e_1 and e_2 are the void ratios corresponding to the respective effective stresses P_1 and P_2.

9.1.2.4 — Method four: overburden pressure swell test procedures

$$\gamma_{h\ swell} = \frac{\Delta H/H}{1.7 - \log_{10} P}$$

where $\Delta H/H$ is the decimal change of specimen eight divided by the initial height; and P is the overburden pressure in psi.

9.1.2.5 — For methods two, three, and four, convert $\gamma_{h\ swell}$ to $\gamma_{h\ shrink}$ using Fig. 9.9.

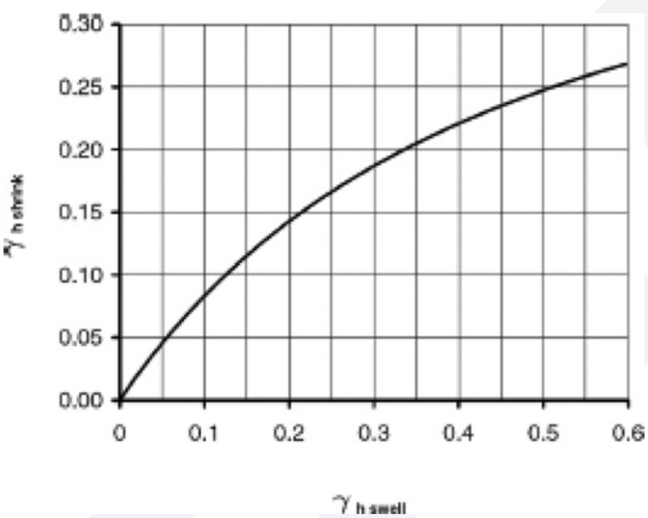

Fig. 9.9—Suction compression index relationship between shrinkage and swelling.

9.1.3 — Modified unsaturated diffusion coefficient α′

For each distinct soil layer described in Section 9.1.1, calculate modified unsaturated diffusion coefficient α' for swelling and shrinkage as follows:

For swelling (edge lift)

$$\alpha'_{swell} = (0.0029 - 0.000162S_s - 0.0122\gamma_{h\ swell})F_f$$

COMMENTARY

C9.1.2.4 — Method four: overburden pressure swell test procedures

To a lesser extent, the overburden swell pressure test also requires undisturbed samples and an effort approximately equivalent to the hydrometer and Atterberg limits procedures.

C9.1.3 — Modified unsaturated diffusion coefficient α′

One modified unsaturated diffusion coefficient α' is calculated for $\gamma_{h\ swell}$ and another coefficient α' is calculated for $\gamma_{h\ shrink}$. The unsaturated diffusion coefficient is also modified by the soil fabric factor, ranging from 1.0 to 1.2, which takes into account the presence of horizontal and vertical moisture flow paths, including roots, desiccation cracks, layers, fractures, and joints.

RECOMMENDATIONS

For shrinkage (edge drop)

$$\alpha'_{shrink} = (0.0029 - 0.000162S_s - 0.0122\gamma_{h\ shrink})F_f$$

where F_f is determined from Table 9.1 and

Table 9.1—Soil fabric factor F_f

	Condition	F_f
	Non-CH soils	1.0
CH soils	Profile with one root, crack, sand/silt seam all ≤1/8 in. width/dimension in any combination	1.0
	Profile with two to four roots, cracks, sand/silt seams all larger than 1/8 in. width/dimension in any combination	1.1
	Profile with more than four roots, cracks, sand/silt seams all larger than 1/8 in. width/dimension in any combination	1.2

$$S_s = -20.29 + 0.1555(\text{LL}) - 0.117(\text{PI}) + 0.0684(\%_{-\#200})$$

9.1.4 — Weighted average of α′

For layered soils, calculate α′ for swelling and shrinkage for each layer down to 9 ft (2.7 m) (or more, if justified by geotechnical analysis). Divide the total soil profile into three sections: the top third, the middle third, and the bottom third. Soil layers (or parts of layers) within the top, middle, and bottom thirds of the soil profile shall be assigned a weighting factor of 3, 2, and 1, respectively. The weighted average of α′ shall be determined for each swell mode as the sum of the products of the weighting factor, times the thickness of the layer (or part of layer), times the value of α′ for that layer, divided by the sum of the products of the weighting factor, times the thickness of the layer (or part of layer).

$$(\alpha)_{weighted} = \left(\sum F_i \times D_i \times \alpha_i\right) / \left(\sum F_i \times D_i\right)$$

9.1.5 — Determination of e_m

Determine e_m for edge drop and edge lift swell modes from Fig. 9.10, using a larger value from I_m or α′ charts (using weighted α′ as described in Section 9.1.4). The procedure limits e_m to a maximum of 9 ft (2.7 m) for any case of edge drop or edge lift.

COMMENTARY

C9.1.4 — Weighted average of α′

The weighting protocol is described in Section 3.2.9 of PTI DC10.1-08.[2] A specific example, with calculations, is presented in Section 3.6.3 of the same document.

For layered soils, weighted averages of several soil properties must be calculated. This document requires weighted averages for the PI, the suction compression index γ_h for both swelling and shrinking conditions (that is, $\gamma_{h\ swell}$ and $\gamma_{h\ shrink}$), and the modified unsaturated diffusion coefficient α′. The procedure for calculating the weighted average of all the soil properties is the same.

RECOMMENDATIONS | **COMMENTARY**

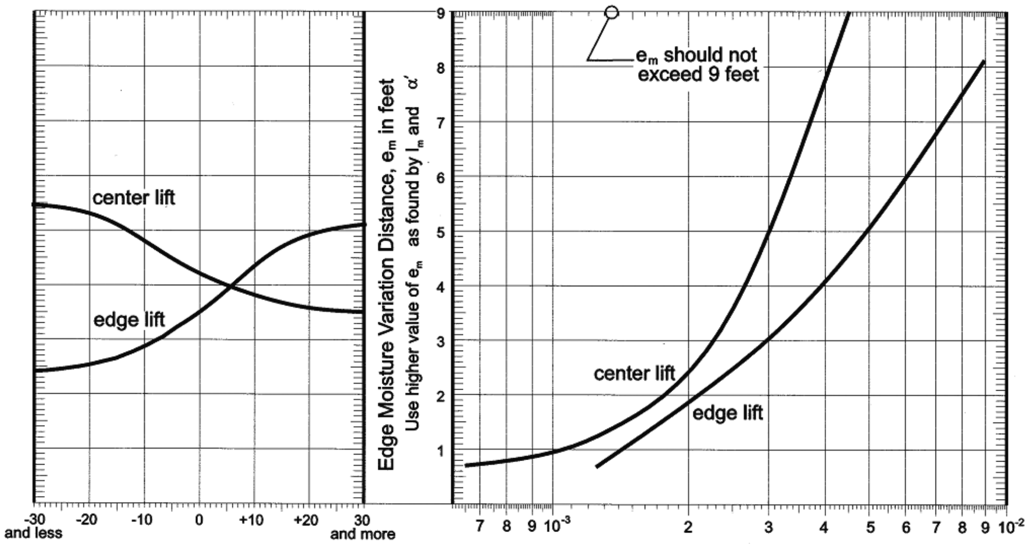

Fig. 9.10—Edge moisture variation distance e_m *selection chart.*

9.2 — Differential soil movement y_m

9.2.1 — Determination of y_m by computer methods

Differential soil movement y_m may be determined by computer methods, or for those cases where the soil suction changes are controlled by normal environmental influences (including proper irrigation practices); y_m shall be determined using the stress change factors (SCFs) in Table 9.2(a) post-equilibrium suction envelope) or Table 9.2(b) (post-construction suction envelope). Tables 9.3(a), (b), (c), and (d) provide SCFs for selected nonenvironmental influences. Other nonenvironmental influences, such as tree removal, poor drainage, high water tables, shallow rock, soil conditioning, and so on, require modeling by computer methods.

These SCF tables assume the depth to constant suction is 9 ft (2.7 m) and γ_h of the soil layers does not vary by more than 10%. If these assumptions are not appropriate, computer methods shall be used.

C9.2.1 — Determination of y_m by computer methods

The SCF method should only be used if a typical trumpet-shaped final suction profile as shown in Fig. C9.1 can be assumed, the depth to constant suction can be assumed to be 9 ft (2.7 m), and γ_h does not vary by more than 10% between layers in the soil profile. Otherwise, this method may not be accurate.

For nonstandard design conditions where these assumptions are not appropriate, (VOLFLO),[12] a commercially available computer program, may be used to determine y_m in accordance with Section 9.2.1.

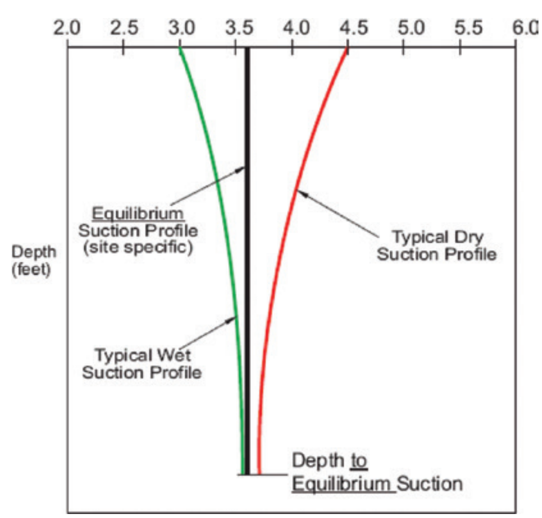

Fig. C9.1—Soil suction (pF).

RECOMMENDATIONS **COMMENTARY**

Table 9.2(a)—Stress change factor (SCF) for use in determining y_m: post-equilibrium case

	Final controlling suction at surface, *pF*						
Equlibrium suction	2.5	2.7	3.0	3.5	–4.0	–4.2	4.5
2.7	+3.2	0	–4.1	–13.6	–25.7	–31.3	–40.0
–3.0	+9.6	+5.1	0	–7.5	–18.2	–23.1	–31.3
–3.3	+17.7	+12.1	+5.1	–2.6	–11.5	–15.8	–23.1
–3.6	+27.1	+20.7	+12.1	+1.6	–5.7	–9.4	–15.8
–3.9	+38.1	+30.8	+20.7	+7.3	–1.3	–4.1	–9.4
–4.2	+50.4	+42.1	+30.8	+14.8	+3.2	0	–4.1
–4.5	+63.6	+54.7	+42.1	+23.9	+9.6	+5.1	0

Notes: z_m = 9 ft (2.7 m); post-equilbrium case, which is recommended for use for areas of Thornthwaite indexes that are more negative than –15 and more positive than +15; shaded boxes represent extreme cases; atypical trumpet-shaped suction envelopes or depths to equilibrium suction, which may vary from 9 ft (2.7 m), require use of computer analysis.

Table 9.2(b)—Stress change factor (SCF) for use in determining y_m: post-construction case

Suction change *pF*	1.3	1.4	**1.5**	1.6	1.7	1.8	1.9	2.0
Wetting (swelling)	33.2	36.7	**40.2**	43.9	47.6	51.4	55.3	59.2
Drying (shrinking)	–24.3	–26.7	**–29.2**	–31.7	–34.2	–36.7	–39.3	–41.9

Notes: Suction change of 1.5*pF* is recommended. This value has been found to produce designs that are typical and perform well in slab-on-ground design practice. Other values of suction change are listed, which LDPs may use for special cases or different local practices; z_m = 9 ft (2.7 m); Table 9.2(b) is based on post-construction case, which is recommended for areas of Thornthwaite indexes, including and between –15 and +15; atypical trumpet-shaped suction envelopes or depths to equilibrium section, which may vary from 9 ft (2.7 m), require use of computer analysis.

Table 9.3(a)—Stress change factor (SCF) for use in determining y_m: lawn irrigation

Equilibrium suction (*pF*) at depth z_m	Stress change factor							
	Controlling surface suction due to lawn watering							
	pF, units				With 4 ft (1.2 m) deep moisture barrier *pF*, units			
pF	2.5	2.7	3.0	3.5	2.5	2.7	3.0	3.5
2.7	3.2	0	0	0	0.1	0	0	0
3.0	9.6	5.1	0	0	0.1	0.1	0	0
3.3	17.7	12.1	5.1	0	0.1	0.1	0.1	0
3.6	27.1	20.7	12.1	1.6	1.3	0.5	0.1	0.1
3.9	38.1	30.8	20.7	7.3	3.8	1.9	0.5	0.1
4.2	50.4	42.1	30.8	14.8	7.7	4.9	1.9	0.1
4.5	63.6	54.7	42.1	23.9	12.4	9.1	4.9	0.8

Table 9.3(b)—Stress change factor (SCF) for use in determining y_m: flower bed case (4 ft [1.2 m] deep flower bed moisture)

Equilibrium suction (*pF*) at depth z_m	Stress change factor						
	Controlling surface suction due to flower bed						
	pF, units				With 4 ft (1.2 m) deep moisture barrier *pF*, units		
pF	2.5	3.0	3.5	2.5	2.7	3.0	3.5
2.7	3.2	0	0	0	0	0	0
3.0	13.1	7.0	0	0	0	0	0
3.3	27.3	14.2	0	3.7	1.0	0	0
3.6	48.7	35.1	1.6	11.6	6.2	1.1	0
3.9	69.5	35.1	10.2	22.5	15.2	6.4	0
4.2	90.3	56.0	21.5	35.1	26.6	15.3	2.4
4.5	110.0	76.7	42.3	49.0	39.7	26.6	9.1

RECOMMENDATIONS

Table 9.3(c)—Stress change factor (SCF) for use in determining y_m: tree drying case (without moisture barrier)

Depth of tree root zone, ft	Stress change factor						
	Measured equilibrium suction at depth, z_m, *pF* units						
	2.7	3.0	3.3	3.6	3.9	4.2	4.5
4	–79.1	–60.1	–43.2	–28.4	–15.6	–0.1	0.0
10	–169.6	–146.3	–124.9	–82.8	–42.6+	–9.7≠	0.0
15	–244.7	–213.6	–182.5	–108.1*	–42.6+	–9.7≠	0.0
20	–333.4	–292.9	–252.5	–108.1*	–42.6+	–9.7≠	0.0

*Movement active zone, z_A = 11.5 ft
+Movement active zone, Z_A = 7.5 ft
≠Movement active zone, Z_A = 3.5 ft

Table 9.3(d)—Stress change factor (SCF) for use in determining y_m: tree drying case with 4 ft deep moisture barrier

Depth of tree root zone, ft	Stress change factor						
	Measured equilibrium suction at depth, z_m, *pF* units						
	2.7	3.0	3.3	3.6	3.9	4.2	4.5
4	–36.5	–25.2	–15.8	–8.1	–2.6	0.0	0.0
10	–116.3	–102.4	–88.4	–53.1	–21.5+	0.0	0.0
15	–193.5	–170.5	–147.5	–78.5*	–21.5+	0.0	0.0
20	–278.2	–246.1	–214.2	–78.5*	–21.5+	0.0	0.0

*Movement active zone, Z_A = 11.5 ft
+Movement active zone, Z_A = 7.5 ft
≠Movement active zone, Z_A = 3.5 ft

9.2.1.1 — Geographical areas with $I_m < –15$ or $I_m >$ + 15 shall use the post-equilibrium suction envelope. $y_{m\ shrink}$ is calculated using a suction change envelope starting from the equilibrium suction profile to a dry suction profile. $y_{m\ swell}$ is calculated for a suction change envelope starting from the equilibrium suction profile to a wet suction profile.

Unless determined from suction testing or experience, the following surface suction values shall be used:

(a) Equilibrium suction shall be determined from Fig. 9.11.

(b) The surface suction value for the dry suction profile shall be 4.5*pF*.

(c) The surface suction value for the wet suction profile shall be 3.0*pF*.

9.2.1.2 — Geographical areas with $–15 \leq I_m \geq +15$ shall use the post-construction suction envelope with a total suction change at the surface of 1.5*pF*. $y_{m\ shrink}$ is calculated using a suction change envelope starting from a wet suction profile to a dry suction profile. $y_{m\ swell}$ is calculated for a suction change envelope starting from the dry suction profile to a wet suction profile.

COMMENTARY

9.2.1.1 — The surface soil suction values presented should be used for design unless laboratory testing or experience indicates that other values should be used.

(a) 4.5*pF* is the dry suction value representative of the wilting point of vegetation and should be used for normal design conditions. A value of 6.0*pF* is an extreme upper bound representing long-term sunbaked bare ground and should not be used for typical design conditions.

(b) 3.0*pF* is the wet suction value representative of a well-drained site and should be used for normal design conditions. A 2.5*pF* is an extreme suction value that may be used to model long-term saturation conditions and should not be used for typical design conditions.

RECOMMENDATIONS | **COMMENTARY**

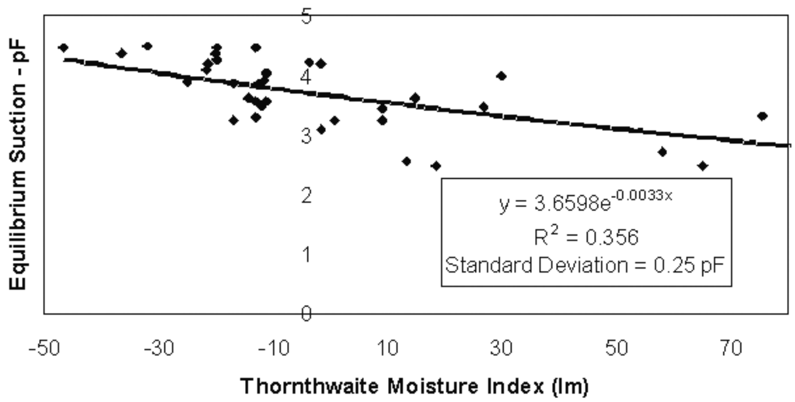

Fig. 9.11—Thornthwaite index–equilibrium suction correlation: correlation is based on data from ASTM A185/A185M and References 13, 15, and 16.

Unless determined from suction testing or experience, the following surface suction values shall be used:

(a) The surface suction value for the dry suction profile shall be 4.5*pF*.

(b) The surface suction value for the wet suction profile shall be 3.0*pF*.

9.2.2 — Determination of y_m by other methods
In lieu of computer methods, it shall be permitted to calculate y_m as follows:

C9.2.2 — Determination of y_m by other methods
This method should only be used if a typical trumpet-shaped final suction profile can be assumed, and γ_h does not vary by more than 10% between layers in the soil profile. Otherwise, this method may not be accurate. Table 9.2(a) assumes the initial suction to be at equilibrium from depth z_m to the ground surface, then becoming wet or dry. This limitation would not yield accurate or conservative results in the case of a dry or wet initial suction profile followed by significant wetting or drying, tree effects, or other moisture anomalies.

RECOMMENDATIONS

9.2.2.1 — For layered soils, calculate a weighted γ_h value $\gamma_{h\,mod}$ for swelling and shrinkage for each layer down to 9 ft (2.7 m) (or more, if justified by geotechnical analysis). Divide the total soil profile into three sections: the top third, the middle third, and the bottom third. Soil layers (or parts of layers) within the top, middle, and bottom thirds of the soil profile shall be assigned a weighting factor of 3, 2, and 1, respectively. $\gamma_{h\,mod\,swell}$ and $\gamma_{h\,mod\,shrink}$ shall be determined as the sum of the products of the weighting factor times the thickness of the layer (or part of the layer), times the value of gh for that layer, divided by the sum of the products of the weighting factor, times the thickness of the layer (or part of layer). y_m for each soil-structure distortion mode shall be taken as

$$y_{m\,swell} = \gamma_{h\,mod\,swell}\,(SCF)$$

$$y_{m\,shrink} = \gamma_{h\,mod\,shrink}\,(SCF)$$

9.2.2.2 — If γ_h varies by more than 10%, a computer modeling program is required to accurately calculate y_m. Nonexpansive layers shall be modeled using γ_h equal to 0.01.

9.3 — Moisture barriers

It shall be permitted to use vertical and horizontal moisture barriers to reduce the soil parameters e_m and y_m if the barriers are designed and installed to mitigate moisture migration to or from the entire perimeter of the foundation area on a permanent basis.

Both vertical and horizontal barriers shall be protected to minimize damage and maintain the integrity of the barrier.

COMMENTARY

C9.3 — Moisture barriers

The effect of a barrier on e_m and y_m may be estimated by the principles of unsaturated soil mechanics.

Conditions can exist, such as desiccated clays; large vertical cracks; nonhomogeneous subsurface conditions (sand layers and so on); site slope; or vertical moisture movements, which may minimize or eliminate the effect of a vertical and/or horizontal barrier. The effect of all barriers should be evaluated by an LDP.

Table 9.4(a)—Value of reduced e_m for various perimeter vertical moisture barriers for CH soils

		Depth of barrier, ft						
		2.0	2.5	3.0	3.5	4.0	4.5	5.0
e_m, ft (center or edge)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	3.0	2.2	2.0	2.0	2.0	2.0	2.0	2.0
	4.0	3.5	3.1	2.0	2.0	2.0	2.0	2.0
	5.0	4.6	4.3	4.0	2.8	2.5	2.5	2.5
	6.0	5.7	5.5	5.2	4.2	3.0	3.0	3.0
	7.0	6.7	6.5	6.3	5.5	4.5	3.5	3.5
	8.0	7.7	7.6	7.4	6.6	5.7	4.7	4.0
	9.0	8.8	8.6	8.5	7.7	6.9	6.0	4.9

RECOMMENDATIONS **COMMENTARY**

Table 9.4(b)—Value of reduced e_m for various perimeter vertical moisture barriers for non-CH soils

		Depth of barrier, ft						
		2.0	2.5	3.0	3.5	4.0	4.5	5.0
e_m, ft (center or edge)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	3.0	2.2	2.0	2.0	2.0	2.0	2.0	2.0
	4.0	3.5	3.1	2.6	2.0	2.0	2.0	2.0
	5.0	4.6	4.3	4.0	2.8	2.0	2.0	2.0
	6.0	5.7	5.5	5.2	4.2	3.0	2.0	2.0
	7.0	6.7	6.5	6.3	5.5	4.5	3.2	2.0
	8.0	7.7	7.6	7.4	6.6	5.7	4.7	3.3
	9.0	8.8	8.6	8.5	7.7	6.9	6.0	4.9

Table 9.4(c)—Value of reduced e_m for various perimeter horizontal moisture barriers for CH soils

		Width of barrier, ft								
		2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5
e_m, ft (center or edge)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	3.0	2.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	4.0	3.5	3.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	5.0	4.5	4.0	3.5	3.0	2.5	2.5	2.5	2.5	2.5
	6.0	5.5	5.0	4.5	4.0	3.5	3.0	3.0	3.0	3.0
	7.0	6.5	6.0	5.5	5.0	4.5	4.0	3.5	3.5	3.5
	8.0	7.5	7.0	6.5	6.0	5.5	5.0	4.5	4.0	4.0
	9.0	8.5	8.0	7.5	7.0	6.5	6.0	5.5	5.0	4.5

Table 9.4(d)—Value of reduced e_m for various perimeter horizontal moisture barriers for non-CH soils

		Width of barrier, ft												
		2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	7.5	8.0	8.5
e_m, ft (center or edge)	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	3.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	4.0	3.1	2.6	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	5.0	4.3	4.0	2.8	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	6.0	5.5	5.2	4.2	3.0	2.0	2.0	2.5	2.0	2.0	2.0	2.0	2.0	2.0
	7.0	6.5	6.3	5.5	4.5	3.2	4.0	3.5	3.0	2.5	2.0	2.0	2.0	2.0
	8.0	7.6	7.4	6.6	5.7	4.7	3.3	4.5	4.0	3.5	3.0	2.5	2.0	2.0
	9.0	8.5	8.0	7.5	7.0	6.5	6.0	5.5	5.0	4.5	4.0	3.0	3.0	2.5

Note: 1 ft = 0.30 m.

RECOMMENDATIONS

For CH soil, e_m or y_m with barriers shall not be less than 50% of the e_m or y_m, respectively, without barriers. e_m with barriers shall not be less than 2 ft (0.6 m).

For non-CH soil, e_m or y_m with barriers shall not be less than 25% of the e_m or y_m, respectively, without barriers. e_m with barriers shall not be less than 2 ft (0.6 m).

9.3.1 — Vertical barriers
In lieu of computer methods, the effect of a vertical barrier on e_m shall be obtained by using either Table 9.4(a) or 9.4(b).

A vertical barrier shall extend a minimum of 2 ft (0.6 m) below the adjacent ground surface to be considered to have an effect on e_m and y_m. y_m shall not be less than 80% of the y_m without barriers for a vertical barrier less than 3 ft (0.9 m).

9.3.2 — Horizontal barriers
In lieu of computer methods, the effect of a horizontal barrier on em shall be obtained by using Table 9.4(c) or 9.4(d).

A horizontal barrier shall extend a minimum of 2.5 ft (0.76 m) away from the foundation system to be considered to have an effect on e_m and y_m.

$$e_m \text{ (with barrier)} = e_m \text{ (without barrier)} - \text{(width of barrier} - 2 \text{ ft [0.6 m])}$$

Horizontal barriers shall be protected against damage that would reduce the effectiveness of the barrier.

COMMENTARY

C9.3.2 — Horizontal barriers
The effect of the barrier on y_m requires the use of a two-dimensional (2-D) moisture-flow analysis computer program, such as VOLFLO.[12]

Local conditions may dictate a wider and deeper minimum, and the LDP should account for factors discussed in Section C9.3.

Horizontal barriers may be protected by an above-ground or below-ground protection layer, such as concrete, asphalt, or pavers.

10.0 — MATERIALS

C10.0 — MATERIALS

10.1 — Concrete

10.1.1 — Concrete shall have a minimum specified compressive strength of 2500 psi (17 MPa) at 28 days.

10.1.2 — Admixtures containing calcium chloride shall not be used.

RECOMMENDATIONS

10.2 — Reinforcement

10.2.1 — Prestressed reinforcement

10.2.1.1 —Tendons shall conform to PTI M10.6-15.[13]

10.2.1.2 — Allowable stresses

(a) At jacking force, tensile stress shall not exceed $0.94f_{py}$ or $0.80f_{pu}$.

(b) Immediately after prestress transfer, tensile stress at anchorage devices shall not exceed $0.70f_{pu}$.

10.2.2 — Non-prestressed reinforcement

10.2.2.1 — Deformed reinforcement shall conform to ASTM A615/A615M, Grade 40 or 60, or ASTM A706/A706M.

10.2.2.2 — Welded-wire reinforcement shall conform to ASTM A185/A185M.

10.2.3 — Cover to reinforcement

Minimum concrete cover to tendons (excluding anchors and strand tails) and non-prestressed reinforcement shall be as follows:

10.2.3.1 — Ribs

Top: 1 in. (25 mm)

Bottom: 3 in. (76 mm)

Sides: 2.5 in. (64 mm)

10.2.3.2 — Slabs (including uniform thickness foundation [UTF])

Top: 1 in. (25 mm)

Bottom: 1.5 in. (38 mm)

10.3 — Anchors

Bearing stresses on concrete created by anchors shall not exceed:

COMMENTARY

C10.3 — Anchors

The constant has been increased for slab-on-ground construction from 1.25 to 1.40 at transfer to allow for stressing of the tendons at a minimum concrete compressive strength of 2000 psi (14 MPa). Experience has shown that this is an acceptable practice, provided that the anchors

RECOMMENDATIONS

At transfer of prestress force

$$f_{bp} = 0.8f'_{ci}\sqrt{\frac{A'_b}{A_b} - 0.2} \leq 1.40f'_{ci}$$

where actual bearing stress is

$$\frac{P_i}{n_t A_b}$$

After all prestress losses

$$f_{bp} = 0.6f'_c\sqrt{\frac{A'_b}{A_b}} \leq f'_c$$

where actual bearing stress is

$$\frac{P_e}{n_t A_b}$$

10.4 — Durability

10.4.1 — Foundation concrete exposed to freezing and thawing or to deicing chemicals shall have a minimum specified compressive strength of 3000 psi (21 MPa) at 28 days.

10.4.2 — Concrete in direct contact with soil containing water-soluble sulfates or chlorides shall conform to the following:

10.4.2.1 — Soil sulfates

10.4.2.1.1 — For soil sulfate concentrations greater than or equal to 0.1% but less than 0.2% by weight, concrete shall be made with Type II or V cement.

COMMENTARY

are cast into a perimeter rib or thickened section that is at least 11.5 in. (290 mm) deep, that the anchor is located and oriented such that the square root of A_b'/A_b is greater than 3.2, and that the nominal slab tendon spacing is greater than 24 in. (0.6 m).

Refer to Chacos[14] for further information.

C10.4 — Durability

C10.4.2 — When a moisture control barrier such as a polyethylene vapor retarder is placed between the concrete (including the sides and bottom of the ribs) and the soil, the concrete is not considered to be in direct contact with soil within the context of Section 10.4.

RECOMMENDATIONS

COMMENTARY

10.4.2.1.2 — For soil sulfate concentrations equal to or greater than 0.2% by weight, concrete shall be made with Type V cement (or approved equivalent) and shall have a minimum compressive strength of 3000 psi (21 MPa) at 28 days.

10.4.2.1.3 — Concentrations of water-soluble soil sulfates shall be determined by California Department of Transportation Test 417,15 or another current test method recognized in the governing building code or commonly used in the geographic area of the project.

10.4.2.2 — Soil chlorides

When concrete is in direct contact with soil containing a level of chloride ions that is known to have caused tendon failure due to corrosion in the local area as determined by local experience and practice, tendons and reinforcing steel shall be protected from corrosion according to Sections 10.4.2.2.1, 10.4.2.2.2, or 10.4.2.2.3.

C10.4.2.2 — Soil chlorides

Concentrations of soil chloride ions can be determined by California Department of Transportation Test 422,[16] or another current test method recognized in the governing building code or commonly used in the geographic area of the project.

10.4.2.2.1 — Use minimum concrete cover in accordance with Table 10.1.

C10.4.2.2.1 — Table 4.1 is derived from Table 8.22.1 of the California Department of Transportation's "Bridge Design Specifications."[17]

10.4.2.2.2 — Use encapsulated tendons.

10.4.2.2.3 — Use other means of mitigating corrosion as approved by the LDP.

C10.4.2.2.3 — ACI 222.3R-03[16] describes a variety of techniques that may be used to protect steel embedded in concrete against corrosion.

Table 10.1—Recommended minimum concrete cover (excluding anchors and strand tails) for corrosive soil

	Chloride concentration, ppm		
	500 to 5000	5001 to 10,000	>10,000
Minimum concrete cover	3 in.	4 in.	5 in.

Note: The above minimums are not required if encapsulated tendons are used per Section 4.3.2.2.2 and/or other means of mitigating corrosion are used per Section 4.3.2.2.3, unless otherwise specified.

11.0— REFERENCES

11.1 — Referenced standards and reports

The standards and reports listed as follows were the latest editions at the time this document was prepared. Because these documents are revised frequently, the reader is advised to contact the proper sponsoring group if it is desired to refer to the latest version.

ASTM International

A185/A185M	Standard Specification for Steel Welded Wire Reinforcement, Plain, for Concrete
A615/A615M	Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement
A706/A706M	Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement
D422	Standard Test Method for Particle-Size Analysis of Soils
D4318	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D4546	Standard Test Methods for One-Dimensional Swell or Collapse of Soils
D4829	Standard Test Method for Expansion Index of Soils

International Code Council
International Building Code

These publications may be obtained from the following organizations:

ASTM International
100 Barr Harbor Dr.
West Conshohocken, PA 19428
www.astm.org

International Code Council
500 New Jersey Avenue, NW, 6th Floor
Washington, DC 20001
www.iccsafe.org

11.2 — Cited references

1. ACI Committee 318, "Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318R-14)," American Concrete Institute, Farmington Hills, MI, 2014, 519 pp.

2. PTI Committee DC-10, "Design of Post-Tensioned Slabs-on-Ground (PTI DC10.1-08)," third edition with 2008 supplement, Post-Tensioning Institute, Farmington Hills, MI, 2008, 106 pp.

3. Thornthwaite, C. W., "An Approach Toward a Rational Classification of Climate," *Geographical Review*, V. 38, No. 1, 1948, pp. 55-94.

4. Wray, W. K, "Development of a Design Procedure for Residential and Light Commercial Slabs-on-Ground Constructed Over Expansive Soils," PhD thesis, Texas A&M University, College Station, TX, 1978.

5. Zia, P. H.; Peterson, K.; Scott, N. L.; and Workman, E. B., "Estimating Prestress Losses," *Concrete International*, V. 1, No. 6, June 1979, pp. 32-36.

6. Joint ACI-ASCE Committee 423, "Guide to Estimating Prestress Loss (ACI 423.10R-16)," American Concrete Institute, Farmington Hills, MI, 2016, 68 pp.

7. Bondy, K. B., "Cracking in Post-Tensioned Ground-Supported Slabs on Expansive Soils," *Technical Note #6*, Post-Tensioning Institute, Farmington Hills, MI, Aug. 1995.

8. PTI Committee DC-10, "Design of Post-Tensioned Slabs-on-Ground (PTI DC10.1-08)," second edition, Post-Tensioning Institute, Farmington Hills, MI, 1996, 155 pp.

9. Bondy, K. B., "Performance Evaluation of Residential Concrete Foundations," *Technical Note #9*, Post-Tensioning Institute, Farmington Hills, MI, July 2000.

10. Covar, A. P., and Lytton, R. L., "Estimating Soil Swelling Behavior Using Soil Classification Properties," *Geotechnical Publication*, No. 115, 2001, pp. 44-63.

11. Lytton, R. L., "Prediction of Movement of Expansive Clays," *Geotechnical Special Publication*, No. 40, V. 2, 1994, pp. 1827-1845.

12. VOLFLO 1.5, A Computer Program Available through Geostructural Tool Kit, Inc., Austin, TX, 2005, http://www.gtksoft.com/volflo.htm.

13. PTI Committee M-10, "Specification for Unbonded Tendons for Slab-on-Ground Construction (PTI M10.6-15)," Post-Tensioning Institute, Farmington Hills, MI, 2015, 35 pp.

14. Chacos, G. P., "Back-Up Bars for Residential Slab-on-Ground Foundations," *PTI JOURNAL*, V. 5, No. 1, July 2007, pp. 17-22.

15. "Method of Testing Soils and Waters for Sulfate Content, California Test 417," State of California, Department of Transportation, Engineering Service Center, Sacramento, CA, Mar. 1999.

16. "Method for Testing Soils and Waters for Chloride Content, California Test 422," State of California, Department of Transportation, Engineering Service Center, Sacramento, CA, Apr. 2000.

17. "Bridge Design Specifications," State of California, Department of Transportation, Sacramento, CA, Sept. 2003.

18. ACI Committee 222, "Design and Construction Practices to Mitigate Corrosion of Reinforcement in Concrete Structures (ACI 222.3R-03)," American Concrete Institute, Farmington Hills, MI, 2003, 29 pp.

APPENDIX

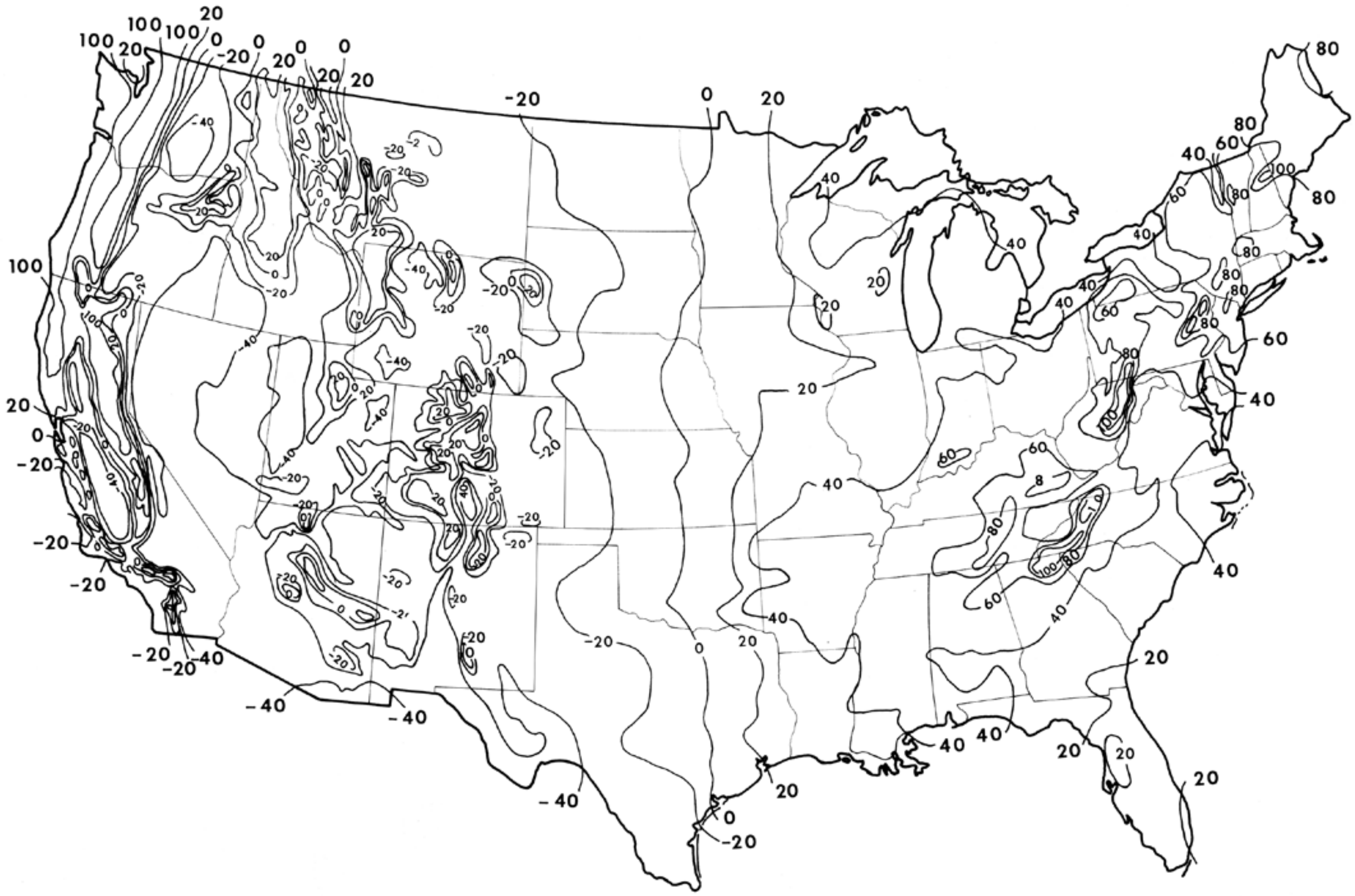

Fig. A1—Thornthwaite moisture index distribution in the United States.

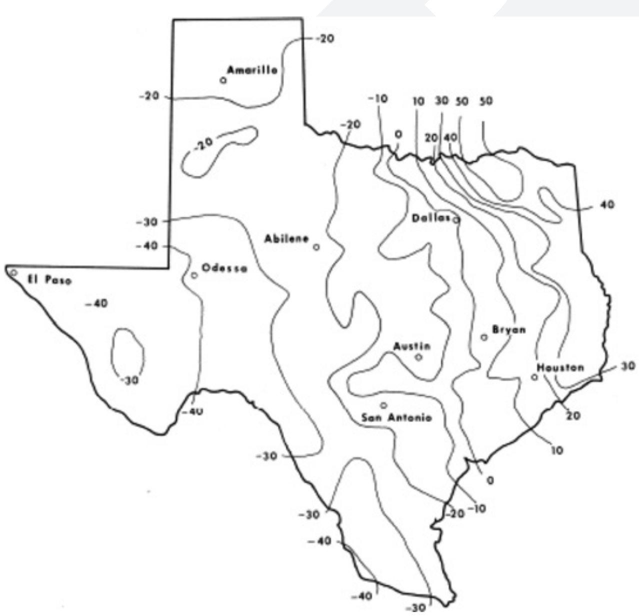

Fig. A2—Thornthwaite moisture index for Texas (20-year average; 1955 to 1974).

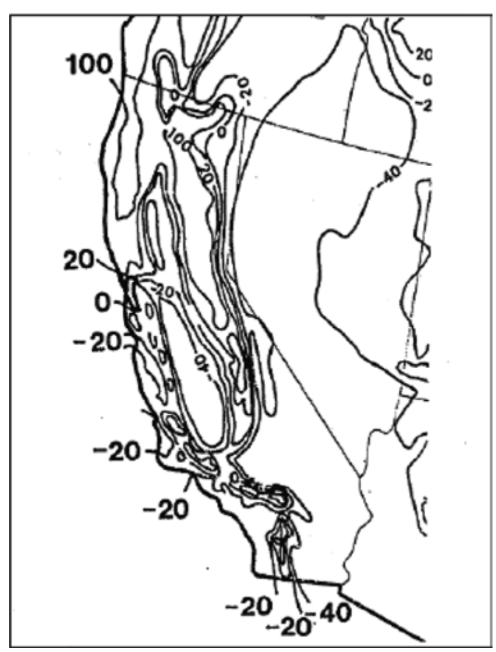

Fig. A3—Thornthwaite moisture index distribution in California.

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- Technical and certification committees that provide consensus guides, reports, manuals, specifications, standards, and certification manuals
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