RECOMMENDATIONS

Change Title to: DC10.5-xx: Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive and Stable Soils

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 4.1.

COMMENTARY

R1.0 — 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.
RECOMMENDATIONS

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 318-141. 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 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.
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.

**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.

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

**Non-compliant 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.

**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.

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Fig. R2.2 — *Primary design rectangle example*

A given design may include multiple primary design rectangles.

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

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.

**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.

**Ribbed foundation** – a foundation system consisting of a uniform thickness slab with ribs that satisfy the requirements of Section 6.2.2 and project downward from the bottom of the slab in both directions. The slab and ribs are considered to act monolithically.
**RECOMMENDATIONS**

**Secondary design rectangle** – a design rectangle which includes specific portions of the foundation which extend outside the limits of the primary design rectangle.

**COMMENTARY**

*Figure R2.3 – 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 judgement as well as past experience on the design of the interface of these appendages.
RECOMMENDATIONS

Stiffness – for purposes of this standard, product of $E_I$ 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

3.0 — 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$^2$
**RECOMMENDATIONS**

\( A_0 \) = coefficient in equation for \( M_L \)

\( A_{ps} \) = total cross-sectional area of prestressed reinforcement, \( \text{in}^2 \)

\( A_s \) = total cross-sectional area of non-prestressed reinforcement, \( \text{in}^2 \)

\( A_{sl} \) = total cross-sectional area of slab concrete, \( \text{in}^2 \)

\( A_v \) = area of rib shear reinforcement, \( \text{in}^2 \)

\( B \) = constant used in the equation for \( M_L \)

\( B_w \) = assumed slab width, \( \text{in.} \)

\( b \) = width of individual rib, \( \text{in.} \)

\( C \) = constant used in equation for \( M_L \)

\( C_A \) = coefficient used to establish minimum foundation stiffness

\( CR \) = prestress loss due to creep of concrete, kips

\( c \) = distance between CGC and extreme cross-section fibers, \( \text{in.} \)

\( E_c \) = modulus of elasticity of concrete, psi \( = 57,000 f'_c \)

\( E_{st} \) = long-term or creep modulus of elasticity of concrete, psi

\( E_{il} \) = expansion index

\( ES \) = prestress loss due to the elastic shortening of concrete, kips

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

\( e \) = base of natural (Naperian) logarithms

**COMMENTARY**

Unless specific testing shows a refined value is justified, \( E_{st} \) may be assumed to be \( 0.5 \times E_c \)
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\( e_1, e_2 = \) 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 (\( \alpha \)) for presence of roots, layers, fractures, and joints.

\( f = \) 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

\( f_{cr} = \) concrete compressive strength at time of stressing tendons, psi

\( f_{fr} = \) 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 = \( \frac{1000P_i}{A} \), psi

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

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\( f_{pj} \) = 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  

\( f_l \) = allowable flexural tension stress in concrete, psi  

\( f_r \) = specified yield strength of non-prestressed reinforcement, psi  

\( f_c \) = percentage of fine clay  

\( g \) = moment of inertia factor  

\( H \) = 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  

This index \( I_m \) is derived from agricultural soil science 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.
**RECOMMENDATIONS**

- **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 the edge drop or edge lift; positive if producing tension at bottom of foundation, negative if producing tension at top of the foundation, ft-k/ft
- **M_S** = maximum applied service load moment in short direction from either the edge drop or edge lift; positive if producing tension at bottom of the foundation, negative if producing tension at top of the 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 the exterior wall, excluding any foundation concrete weight, lb/ft
- **PI** = plasticity index, %
- **PL** = plastic limit, %

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\[ P_e = \text{effective prestress force in tendon after losses due to elastic shortening, creep and shrinkage of concrete, and steel relaxation, kips} \]

\[ P_e = P_i - ES - CR - SH - RE \]

\[ P_i = \text{prestress force in tendon immediately after stressing and anchoring tendons considering effects of tendon friction, kips} \]

\[ P_r = \text{effective prestress force in concrete after losses due to tendon friction, elastic shortening, creep and shrinkage of concrete, steel relaxation, and subgrade friction, kips} \]

\[ P_r = P_e - SG \]

\[ P_s = \text{prestress force at jacking end immediately before anchoring tendons, kips} \]

\[ P_1, P_2 = \text{overburden soil pressures corresponding to void ratios } e_1 \text{ and } e_2, \text{ psi} \]

\[ pF = \text{soil suction value expressed as common logarithm of height of water (in cm) that suction energy can support} \]

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.

\[ q_{allow} = \text{allowable soil bearing pressure, lb/ft}^2 \]

\[ q_u = \text{unconfined compressive strength of soil, lb/ft}^2 \]
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RE = prestress loss due to steel relaxation, kips

r; = 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; unit less measure of foundation Irregularity

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

SH = prestress loss due to concrete shrinkage, kips

t = slab thickness in ribbed foundation, in.

V_L = maximum shear force in long direction underservice load from either edge drop or edge lift, kips/ft

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

v = applied shear stress under service load, psi

v_c = allowable shear stress in concrete, psi
RECOMMENDATIONS

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/ft³

y_m = maximum unrestrained differential soil movement, in.

\beta = \text{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{\frac{E_{c} I}{1000}}

y_m\text{ shrink} = y_m \text{ value for edge drop, in.}

y_m\text{ swell} = y_m \text{ value for edge lift, in.}

z = \text{smaller of L or 6} \beta \text{ in direction considered, ft}

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

If the soil beneath the slab experiences a change in its moisture content after construction of the slab, it will distort into either a edge drop mode (also termed “edge drying,” “center heave,” “edge drop,” 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 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 \beta—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 \beta.

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...
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- \( \alpha = \) unsaturated diffusion coefficient: measure of moisture movement in unsaturated soils

- \( \alpha' = \) unsaturated diffusion coefficient modified by soil fabric factor: \( \alpha' = \alpha F_f \)

- \( \alpha'_{\text{shrink}} = \alpha' \) value for edge drop

- \( \alpha'_{\text{swell}} = \alpha' \) value for edge lift

- \( \gamma_h = \) change of soil volume for unit change in suction corrected for actual percentage of fine clay; also referred to as matrix suction compression index

- \( \gamma_{h_{\text{mod}}} = \gamma_h \) weighted for layered soils

- \( \gamma_{h_{\text{mod shrink}}} = \gamma_{h_{\text{mod}}} \) value for center lift

- \( \gamma_{h_{\text{mod swell}}} = \gamma_{h_{\text{mod}}} \) value for edge lift

- \( \gamma_0 = \) change of soil volume for unit change in suction for 100% fine clay

- \( \mu = \) coefficient of friction between foundation and subgrade

**COMMENTARY**

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.

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.
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 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.

R4.1.1 — 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 provides examples of the overlapping rectangle method.

The Shape Factor (SF) is the perimeter of the contiguous slab squared divided by the area of the contiguous slab.

\[ SF = \frac{(\text{foundation perimeter})^2}{\text{foundation area}} \]

where the foundation perimeter is measured in feet; and the foundation area is measured in ft².

The Simplified Shape Factor (SSF) is 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.

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.
RECOMMENDATIONS

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 ym-center to less than 2.0 in. (5.08 cm) and ym-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.

COMMENTARY

R4.1.2 — Perimeter load

The mathematical analysis forming the basis for the equations for internal forces and deflections in this standard14 consider perimeter loads between 600 and 1500 lb/ft. 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.
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 \( P_r \), SG shall be calculated as follows:

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

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

\[ SG = \left( \frac{W_{slab}}{2000} \right) \left( \frac{\beta}{L} \right) (\mu) \]

where \( \beta \) and \( L \) are in the direction being considered.

R4.1.3 — Concentrated loads

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

R4.1.4 — 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 \( \beta \)-length inward from the edge of the foundation.

\[ ES, CR, SH, \text{ and } RE \] can be calculated with generally accepted methods for estimating losses in prestressed concrete. Total prestress loss (after the effects of tendon friction) is the sum of \( ES, CR, SH, \text{ and } RE \). 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.

The expression for \( P_t \) assumes a high-side friction “wobble” coefficient of 0.002 (refer to ACI 318-08, Table R18.6.2), and one-end tendon stressing (that is, \( P_t \) is assumed to act at the far end of the tendon). In lieu of more...
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detailed analysis, a value of \( P_l = P_s/(1 + 0.002L) \) may be used. Typically, \( P_s = 0.8 A_{ps} f_{pu} \).

\( SG \) 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 \( \beta \) 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.

Two factors were identified to have an important effect on the magnitude of the coefficient of friction \( \mu \). These factors are 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, and the material over which sliding occurs.

Measured slab movements indicate that summertime concrete placement results in effective \( \mu \) values in the range of 0.50 to 0.60 for UTFs cast on polyethylene sheeting. Winter placement, which occurs in the southern climates of the United States, may result in displacements corresponding to coefficients still operating on the “1st movements” curve in Fig. R6.1. The effective coefficient for these conditions ranges between 0.60 and 0.75 for polyethylene sheeting.

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

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. R6.2.

Fig. R4.1—Effect of successive slab movement on Timm’s 5 in. thick slab cast on polyethylene sheeting.
Fig. R4.2—Summary of coefficient of friction for 5 in. slabs.

4.2 — Ribbed foundations

Calculations for ribbed foundations shall be based on criteria specified in Sections 6.2.1 to 6.2.4.

R4.2—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.

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.
4.2.1 — Minimum slab thickness

Minimum slab thickness $t$ shall be 4 in. (10.16 cm).

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. ($[t + 17.78]$ cm) or 11 in. (27.94 cm). 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. (15.24 cm) nor greater than 14 in. (35.56 cm).

R4.2.2.1.1 — 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. 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%.

R4.2.2.1.2 — 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. (15.24 to 35.56 cm). 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. (20.32 cm) wide may be impractical due to excavation considerations. Rib widths greater than 14 in. (35.56 cm) may be used if required for bearing. In that case, however, a width of 14 in. (35.56 cm) shall be used in section property calculations. Excavated rib widths most commonly found in practice are 10 to 12 in. (25.40 to 30.48 cm).
4.2.2.2 — Rib spacing

Rib spacing $S$ used in actual construction shall be a maximum of 15 ft (4.57 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 (1.83 m) nor greater than 15 ft (4.57 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 (kips). $P_r$ shall be determined using the restress at mid-slab or the location of the minimum prestress.

R4.2.2.2 — 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.

R4.2.2.3 — 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.

R4.2.3 — Minimum prestress force for ribbed foundations

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

4.2.4 — Soil-bearing pressure

Applied soil-bearing pressure shall be evaluated using generally accepted techniques and shall not exceed \( q_{\text{allow}} \) 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 4.3.4 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.

4.3.1 — UTF conversion

Minimum thickness shall be

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

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. (19.05 cm) unless a continuous rib, conforming to Section 4.3.2.1, is provided along the entire perimeter.

**COMMENTARY**

R4.2.4 — Soil-bearing pressure

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

R4.3 — 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 4.3.4) 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 cracked section requirements in Section 5.4), shear criteria in Section 6.0, and minimum stiffness requirements in Section 7.0. (Note that \( \beta \) distances can be different in the conformant ribbed foundation and the converted UTF.)

R4.3.1 — 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.
**RECOMMENDATIONS**

4.3.2 — Minimum prestress force for UTFs

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

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.

**COMMENTARY**

R4.3.2 — 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 4.3.3). 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.

R4.3.3 — Soil-bearing pressure

Refer to PTI DC10.1-08\textsuperscript{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

$$f = \frac{1000P_r}{A} + \frac{12,000M_{LS}}{S_{T,B}A} + \frac{1000P_r^2}{S_{T,B}}$$

Maximum moment M shall be as specified in Sections 7.1 and 7.2. $P_r$ shall be calculated at the point of maximum moment, which is at distance $\beta$ from the edge of the slab.

R5.0 — 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\textsuperscript{14} provides background and derivations of the equations specified in Section 5.0.
RECOMMENDATIONS

5.1 — Edge drop

5.1.1 — Long direction

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

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 \( 0 \leq e_m \leq 5 \)

\[ B = 1 \] and \( C = 0. \)

and for \( e_m > 5 \)

\[ B = \left( \frac{y_m - 1}{3} \right) \leq 1.0 \]

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

5.1.1.a Compute \( M_L \) @ \( e_m \) from geotechnical report

5.1.1.b Compute 5 ft (1.5 m) threshold: \( M_L \) @ \( e_m = 5\) 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:

\[ M_{L\_Design} = Max(M_{L\@e_m}, M_{L\@5\,ft}) \]

COMMENTARY

R 5.1.1. 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).
5.1.2 — Short direction

For \( \text{LL}/\text{Ls} \geq 1.1 \)

\[ M_S = \left( \frac{58 + e_m}{60} \right) M_L \]

For \( \text{LL}/\text{Ls} < 1.1 \)

\[ M_L = M_S \]

5.2 — Edge Lift

5.2.1 — Long Direction

\[ M_L = S^{0.1} (he_m)^{0.78} (y_m)^{0.66} \]

\[ \frac{7.2}{L^{0.0065} p^{0.04}} \]

5.2.2 — Short Direction

For \( \text{LL}/\text{Ls} \geq 1.1 \)

\[ M_S = h^{0.35} \left[ \frac{19 + e_m}{57.75} \right] M_L \]

For \( \text{LL}/\text{Ls} < 1.1 \)

\[ M_L = M_S \]
5.3 — Allowable stress

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

\[ f_t = 6 \sqrt{f'_{c}} \]

Tension: \( f_c \) = 0.45 \( f'_{c} \)

5.4 — Cracked sections

Sufficient reinforcement prestressed or non-prestressed in any combination shall be provided to develop 0.5\( M_L \) and 0.5\( M_S \) 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 non-prestressed reinforcement shall be taken as \( A_{sfy} / 2 \).

5.4.2 — Non-prestressed 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 \( 2\beta \).

R5.3 — 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.

R5.4 — 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 0.5\( M_L \) and 0.5\( M_S \). Bondy15 addresses types of cracking and their ramifications in post-tensioned residential foundations.
RECOMMENDATIONS

6.0 — SHEAR

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

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

6.1 — Applied concrete shear stress

6.1.1 — Ribbed foundations

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

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

\[
V_L = \frac{1}{1940} \left( L^{0.09} S^{0.71} H^{0.43} P^{0.44} y_m^{0.96} e_m^{0.93} \right)
\]

For \( y_m \leq 1 \text{ in. (2.54 cm)} \), \( e_m \) should not exceed 5 ft (1.52 m) for shear only.

6.2.2 — Short direction

COMMENTARY

R6.0 — SHEAR

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

For $L/L_s \geq 1.1$

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

For $L/L_s < 1.1$, $V_S = V_L$

For $y_m \leq 1$ in. (2.54 cm), $e_m$ should not exceed 5 ft (1.52 m) for shear only.

6.3 — Edge lift

6.3.1 — Long and short direction

$$V_L = V_S = \frac{L^{0.07} h^{0.4} P^{0.03} e_m^{0.16} y_m^{0.57}}{3S^{0.015}}$$

6.4 — Allowable stress

Applied shear stress $v$ calculated in accordance with Section 8.0 shall not exceed the following

$$v_c = 2.4 \sqrt{f_{cc}'} + 0.2 \left( 1000 \frac{P_s}{A} \right)$$

The effective prestress force $P_s$ shall be determined using the prestress at $\beta$.

R6.4 — Allowable stress

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

$$A_s = \frac{(v - v_c)b}{S} \left( 0.4f_y \right)$$

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).
RECOMMENDATIONS

7.0 — STIFFNESS

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

$$E_{crI} L \text{ or } S = 12,000 M_{L} \text{ or } S I_{S} \text{ or } L C_{\Delta} Z_{L} \text{ or } S$$

COMMENTARY

R7.0 — 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 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 $E_{crI}$ required should be determined for each direction considering both swell modes. The coefficient $C_{\Delta}$ is a function of the type of superstructure material and the swelling condition (edge drop or edge lift).

Bondy\textsuperscript{17} 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 R7.1 requires very high $C_{\Delta}$ values (resulting in very large required stiffness values) when prefabricated roof trusses are used, regardless of the superstructure material. $C_{\Delta}$ values specified in Table R7.1 for prefabricated roof trusses may be waived, and smaller values based on the 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 $C_{\Delta}$ may be used for other superstructure materials listed in Table R9.1 if effective jointing details are used to minimize cracking, such as closely spaced control joints in brick or stucco walls.
8.0 — GENERAL

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

8.1 — Soils

This standard is applicable to foundations built on expansive soils, as defined in Section 4.1.2.

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.

R8.1.2 — Expansive soils

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

Table R7.1—Recommended values of stiffness coefficient $C_A$

<table>
<thead>
<tr>
<th>Superstructure material</th>
<th>Center lift</th>
<th>Edge lift</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood frame without plaster</td>
<td>240</td>
<td>480</td>
</tr>
<tr>
<td>Stucco or plaster</td>
<td>360</td>
<td>720</td>
</tr>
<tr>
<td>Brick veneer</td>
<td>480</td>
<td>960</td>
</tr>
<tr>
<td>Concrete masonry units</td>
<td>960</td>
<td>1920</td>
</tr>
<tr>
<td>Prefabricated roof trusses*</td>
<td>1000</td>
<td>2000</td>
</tr>
</tbody>
</table>

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

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 (1.52 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 (1.52 m) with a PI of 15 or greater.

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 (1.52 m) layers determined using the depth weighting procedures of Section 4.1.2.1, 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 (1.52 m) layers determined using the depth weighting procedures of Section 4.1.2.1, 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 (1.52 m) layers determined using the depth weighting procedures of Section 4.1.2.1, disregarding the 2 ft (0.60 m) or thicker layer provisions.

9.0 — SOIL PARAMETERS

**COMMENTARY**

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
RECOMMENDATIONS

9.1 — 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 $\alpha$. 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 5.1) 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

R9.1.1 — 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 $\mu$m = % – 2$\mu$, expressed as a percentage of the total sample

9.1.1.6 — Percentage of fine clay

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

9.1.2 — Matrix suction compression index $\gamma_h$ For each significant soil layer described in Section 9.1.1, determine $\gamma_h$ 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.

R9.1.2.1.1 — 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 $\gamma_o = 0.01$. Depths greater than 9 ft (2.74 m) may be used if justified by geotechnical analysis.
RECOMMENDATIONS

9.1.2.1.2 — Determine $\gamma_0$ from Fig. 9.2 to 9.7.

COMMENARY

R9.1.2.1.2 — Interpolate between $\gamma_0$ lines. Beyond extreme contour values, use the nearest values for $\gamma_0$. Figures 9.2 through 9.7 were derived from the National Soil Survey Center, USDA.10

Fig. 9.1—Mineral classification chart.
Fig. 9.2—Zone I chart for determining $\gamma_0$. 

![Zone I Chart](image)
Fig. 9.3—Zone II chart for determining $\gamma_o$. 
Fig. 9.4—Zone III chart for determining $\gamma_o$. 
Fig. 9.5—Zone IV chart for determining $\gamma_o$. 
Fig. 9.6—Zone V chart for determining $\gamma_o$. 
Fig. 9.7—Zone VI chart for determining $\gamma_0$.

9.1.2.1.3 — Correct $\gamma_0$ for the actual percentage of fine clays

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

9.1.2.1.4 — Correct $\gamma_h$ for swelling or shrinkage:

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

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

**9.1.2.1.5 — Correction of $\gamma_h$ for coarse-grained soil**

The correction of $\gamma_h$ for coarse-grained soil shall only be used in cases where the percentage retained on the No. 10 sieve is 10% or more.

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

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

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; $\gamma_{moist}$ is the total unit weight of the soil at the soil wet limit around a pF of 2.5 for clay; $\gamma_{in-situ}$ 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 $\gamma_w$ is the unit weight of water.

**COMMENTARY**

R9.1.2.1.5 — Correction of $\gamma_h$ for coarse-grained soil

The formula for $\gamma_h$ 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 $\gamma_h$ 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).

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

The values of $\gamma_{moist}$ and $\gamma_{in-situ}$ 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.

R9.1.2.2 — 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.
\[ \gamma_{h,\text{swell}} = \frac{EI}{1700} \quad \text{and} \]

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

9.1.2.3 — Method three: consolidation-swell pressure test procedure

Use ASTM D4546, Method C

\[ \gamma_{h,\text{swell}} = \frac{(0.7)(C_s)}{(1 + e_2)} \]

\[ C_s = \frac{(e_1 - e_2)}{\log(P_2) - \log(P_1)} \]

Figure 9.8 shows the void ratio versus the overburden pressure.

R9.1.2.3 — Method three: consolidation-swell pressure test procedure

The consolidation-swell pressure test is a lengthy and expensive test, but the results are reasonably reliable.
Fig. 9.8—Void ratio versus overburden pressure.

$C_s$ 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.

R9.1.2.4 — 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.
9.1.2.5 — For methods two, three, and four, convert $\gamma_{h_{\text{swell}}}$ to $\gamma_{h_{\text{shrink}}}$ using Fig. 9.9.

![Suction compression index relationship between shrinkage and swelling](image)

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

9.1.3 — Modified unsaturated diffusion coefficient $\alpha'$ For each distinct soil layer described in Section 9.1.1, calculate modified unsaturated diffusion coefficient $\alpha'$ for swelling and shrinkage as follows:

For swelling (edge lift)

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

For shrinkage (edge drop)

$$\alpha'_{\text{shrink}}$$

R9.1.3 — Modified unsaturated diffusion coefficient $\alpha'$ One modified unsaturated diffusion coefficient $\alpha'$ is calculated for $\gamma_{h_{\text{swell}}}$ and another coefficient $\alpha'$ is calculated for $\gamma_{h_{\text{shrink}}}$. 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.
\[ \alpha'_{\text{shrink}} = (0.0029 - 0.000162S_s - 0.0122\gamma_h\text{swell})F_f \]

where \( F_f \) is determined from Table 9.1 and

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

Table 9.1—Soil fabric factor \( F_f \)

<table>
<thead>
<tr>
<th>Condition</th>
<th>( F_f )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-CH soils</td>
<td>1.0</td>
</tr>
<tr>
<td>Profile with one root, crack, sand/silt seam all ( \leq \frac{1}{8} ) width/dimension in any combination</td>
<td>1.0</td>
</tr>
<tr>
<td>Profile with two to four roots, cracks, sand/silt seams all larger than ( \frac{1}{8} ) width/dimension in any combination</td>
<td>1.1</td>
</tr>
<tr>
<td>Profile with more than four roots, cracks, sand/silt seams all larger than ( \frac{1}{8} ) width/dimension in any combination</td>
<td>1.2</td>
</tr>
</tbody>
</table>

9.1.4 — Weighted average of \( \alpha' \)

For layered soils, calculate \( \alpha' \) for swelling and shrinkage for each layer down to 9 ft (2.74 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 \( \alpha' \) 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 \( \alpha' \) for that layer, divided by the sum of the products of the weighting factor, times the thickness of the layer (or part of layer).

R9.1.4 — Weighted average of \( \alpha' \)

The weighting protocol is described in Section 3.2.9 of PTI DC10.1-08.: 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 \( \gamma_h \) for both swelling and shrinking conditions (that is, \( \gamma_h\text{swell} \) and \( \gamma_h\text{shrink} \)), and the modified unsaturated diffusion coefficient \( \alpha' \). The procedure for calculating the weighted average of all the soil properties is the same.
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 $\alpha'$ charts (using weighted $\alpha'$ as described in Section 9.1.4). The procedure limits $e_m$ to a maximum of 9 ft (2.74 m) for any case of center or edge lift.

\[
(\alpha)_{\text{weighted}} = \frac{(\Sigma F_i \times D_i \times \alpha)}{(\Sigma F_i \times D)}
\]
RECOMMENDATIONS

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

R9.2.1 — 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) postconstruction 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 (2.74 m) and \( \gamma_h \) of the soil layers does not vary by more than 10%. If these assumptions are not appropriate, computer methods shall be used.

The SCF method should only be used if a typical trumpet-shaped final suction profile as shown in Fig. R9.1 can be assumed, the depth to constant suction can be assumed to be 9 ft (2.74 m), and \( \gamma_h \) 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), a commercially available computer program, may be used to determine \( y_m \) in accordance with Section 9.2.1.

![Fig. R9.1—Soil suction (pF).](image)
## RECOMMENDATIONS

<table>
<thead>
<tr>
<th>Equilibrium suction</th>
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<th>3.5</th>
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<td>+23.9</td>
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</tbody>
</table>

Notes: $z_m = 9\text{ ft (2.74 m)}$; post-equilibrium case, which is recommended for use for areas of Thornthwaite index less than +15; shaded boxes represent extreme cases; atypical trumpet-shaped suction envelopes may, require use of computer analysis.

### COMMENTARY

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

<table>
<thead>
<tr>
<th>Suction change $pF$</th>
<th>1.3</th>
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<tr>
<td>Wetting (swelling)</td>
<td>33.2</td>
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<td>Drying (shrinking)</td>
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<td>−26.7</td>
<td>−28.2</td>
<td>−31.7</td>
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</table>

Notes: Suction change of 1.5$pF$ is recommended. This value has been found to produce designs that meet practice. Other values of suction change are listed, which LDPs may use for special cases or different post-construction cases. It is recommended for areas of Thornthwaite indexes, including and before 15, or depths to equilibrium suction, which may vary from 9 ft (2.74 m), require use of computer analysis.

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

<table>
<thead>
<tr>
<th>Equilibrium suction (pF) at depth z_m</th>
<th>Stress change factor</th>
<th>Controlling surface suction due to lawn watering</th>
<th>With 4 ft deep moisture barrier pF, units</th>
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Table 9.3(b)—Stress change factor (SCF) for use in determining $y_m$: flower bed case (4 ft deep flower bed moisture)

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<th>Equilibrium suction (pF) at depth z_m</th>
<th>Stress change factor</th>
<th>Controlling surface suction due to flower bed</th>
<th>With 4 ft deep moisture barrier pF, units</th>
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</thead>
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Table 9.3(c)—Stress change factor (SCF) for use in determining $y_m$: tree drying case (without moisture barrier)
9.2.1.1—Geographical areas with Im ≤ -15 or Im ≥ +15 shall use the post-equilibrium suction envelope. ym shrink is calculated using a suction change envelope starting from the equilibrium suction profile to a dry suction profile. ym 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.

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

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

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

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 4.5pF.

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

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

**R9.2.2** — 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 $\gamma_h$ 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.

9.2.2.1 — For layered soils, calculate a weighted $\gamma_h$ value $\gamma_{h \text{ mod}}$ for swelling and shrinkage for each layer down to 9 ft (2.74 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

![Equilibrium Suction - pF vs Thornthwaite Moisture Index (Im)]

Fig. 9.11—Thornthwaite index–equilibrium suction correlation: correlation is based on data from ASTM A185/A185M and References 4, 6, and 7.
**RECOMMENDATIONS**

Within the top, middle, and bottom thirds of the soil profile shall be assigned a weighting factor of 3, 2, and 1, respectively. $\gamma_{h \text{mod swell}}$ and $\gamma_{h \text{mod shrink}}$ 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 $\gamma$ for that layer, divided by the sum of the products of the weighting factor, times the thickness of the layer (or part of the layer). $y_m$ for each soil-structure distortion mode shall be taken as:

\[
y_{m\text{swell}} = \gamma_{h\text{mod swell}} (SCF)
\]
\[
y_{m\text{shrink}} = \gamma_{h\text{mod shrink}} (SCF)
\]

### 9.2.2.2 — If $\gamma_h$ varies by more than 10%, a computer modeling program is required to accurately calculate $y_m$. Nonexpansive layers shall be modeled using $\gamma_h$ 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.

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 (0.06 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).

**COMMENTARY**

R9.3 — 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.
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).

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

<table>
<thead>
<tr>
<th>$e_m$, ft (center or edge)</th>
<th>Depth of barrier, ft</th>
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</table>

Table 9.4(b)—Value of reduced $e_m$ for various perimeter vertical moisture barriers for non-CH Soils
9.3.2 — Horizontal barriers

In lieu of computer methods, the effect of a horizontal barrier on $e_m$ 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.

---

R9.3.2 — 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 R9.3.

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

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<th>$e_m$, ft (center or edge)</th>
<th>Width of barrier, ft</th>
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Table 9.4(c)—Value of reduced $e_m$ for various perimeter horizontal moisture barriers for CH soils

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<th>$e_m'$, ft (center or edge)</th>
<th>Width of barrier, ft</th>
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Table 9.4(d)—Value of reduced $e_m$ for various perimeter horizontal moisture barriers for non-CH soils

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10. — Materials

10.1 — Concrete

10.1.1 — Concrete shall have a minimum specified compressive strength of 2500 psi (17.24 MPa) at 28 days.

10.1.2 — Admixtures containing calcium chloride shall not be used.

10.2 — Reinforcement

10.2.1 — Prestressed reinforcement

10.2.1.1 — Tendons shall conform to PTI M10.2-00.4

10.2.1.2 — Allowable stresses

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

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

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. (2.54 cm)
Bottom: 3 in. (7.62 cm)
Sides: 2.5 in. (6.35 cm)

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

Top: 1 in. (2.54 cm)
Bottom: 1.5 in. (3.81 cm)

10.3 — Anchors

Bearing stresses on concrete created by anchors shall not exceed:

At transfer of prestress force

$$f_{bp} = 0.8f_{c}^{'} \sqrt{\frac{A_{b}^{'}}{A_{b}}} - 0.2 \leq 1.40f_{c}^{'}$$

where actual bearing stress is

$$\frac{P_{i}}{n_{r}A_{b}}$$

R10.3 — 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 (13.79 MPa). Experience has shown that this is an acceptable practice, provided that the anchors 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 Chacoss for further information.
After all prestress losses

\[ f_{bp} = 0.6 f'_c \sqrt{\frac{A_b'}{A_b}} \leq f'_c \]

where actual bearing stress is

\[ \frac{P_a}{n_f A_b} \]

### 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 (20.7 MPa) at 28 days.

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

**R10.4.2** — 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.

#### 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.

**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
10.4.2.1.3 — Concentrations of water-soluble soil sulfates shall be determined by California Department of Transportation Test 417, 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.

10.4.2.2.1 — Use minimum concrete cover in accordance with Table 4.1.

10.4.2.2.2 — Use encapsulated tendons.

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

R10.4.2.2 — Soil chlorides

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

R10.4.2.2.1 — Table 4.1 is derived from Table 8.22.1 of the California Department of Transportation’s “Bridge Design Specifications.”

R10.4.2.2.3 — ACI 222.3R-03 describes a variety of techniques that may be used to protect steel embedded in concrete against corrosion.
11— 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 Low-Alloy Steel Deformed and Plain 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 Cohesive 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,” American Concrete Institute, Farmington Hills, MI, 200, 519 pp.


