Addendum No. 2

to the

3rd Edition of the Design of Post-Tensioned Slabs-on-Ground

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Summary of Addendum No. 2 Revisions

1. Table 3.1 – Soil Fabric Factor

   Table restructured to clarify that the use of higher Soil Fabric Factors applies only to CH-type soils. Table values have also been revised downward to better reflect design experience.

2. Slab Shape Factor

   Sections 3.8 and 4.5.1 have been rewritten to clarify that a designer has options that may be considered when the slab shape factor exceeds 24.

3. Rib Spacing

   Section 4.5.2.1 has been revised to permit rib spacings less than 6 feet and to clarify what spacing shall be used to compute moments and shears.

4. Rib Width

   Section 4.5.2.3 has been revised to limit rib widths to a range of 6 to 14 in.

5. Uniform Thickness Conversion

   Section 6.12 has been revised to clarify that when converting to an equivalent uniform thickness foundation, the conformant ribbed foundation need not satisfy the cracked section provisions.

6. The Modulus of Elasticity of Soil, E_{soil} in the equation for β has been set as a constant = 1,000. Because this is the only location that the variable, E_{soil} appears in this manual, the definition for E_{soil} has been dropped from the List of Symbols and Notation in Appendix A.1.

The major factor in determining the edge moisture variation distance is the unsaturated diffusion coefficient, α. This, in turn, depends on the level of suction, the permeability, and the cracks in the soil. For the same diffusion coefficient, the e_m value will be larger for the center lift case in which moisture is withdrawn from soil around the perimeter of the foundation. The e_m value will be smaller for the edge lift case in which moisture is drawn beneath the building into drier soil. Roots, layers, fractures or joints in the soil will increase the diffusion coefficient and increase the e_m value for both the edge lift and center lift conditions. Using representative values based on laboratory test results in each layer, the following values are required to determine edge moisture variation distance, e_m:

- Liquid Limit, LL
- Plastic Limit, PL
- Plasticity Index, PI
- Percentage of soil passing No. 200 sieve (%-#200) expressed as a percentage of the total sample.
- Percentage of soil finer than 2 microns (%-2μ) expressed as a percentage of the total sample.
- Percent fine clay, $\%fc = \left(\frac{\% - 2\mu}{\% - \#200}\right)$ x 100

For example: 45% / 80% = 0.56 x 100

= 56, report as 56%

Calculate the unsaturated diffusion coefficient, α

$$\alpha = 0.0029 - 0.000162\,(S_s) - 0.0122\,(\gamma_h)$$

Where:

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

The resulting unsaturated diffusion coefficient, α, for each significant layer should be converted to the modified unsaturated diffusion coefficient, α', using F_f.

$$\alpha' = \alpha\,F_f$$

where F_f is the soil fabric factor from Table 3.1:

The modified unsaturated diffusion coefficient, α', should be calculated for each significant soil layer by the procedure outlined above. One modified unsaturated diffusion coefficient, α', is calculated for $\gamma_{h\ swell}$ and another coefficient, α', is calculated for $\gamma_{h\ shrink}$. Significant soil layers are to a minimum depth of nine feet. Depths greater than nine feet may be used if justified by geotechnical analysis. The evaluation of the edge moisture variation requires using a weighted average of the modified unsaturated diffusion coefficient. The weighting procedure is given in 3.2.9, and an example of the calculations for $\gamma_{h\ mod}$ is given in 3.6.3.

Table 3.1 - Soil Fabric Factor

Condition		Ff
Non CH Soils		1.0
CH Soils	Profile with 1 root, crack, sand/silt seam all less than or equal to 1/8” width/dimension in any combination	1.0
	Profile with 2 to 4 roots, cracks, sand/silt seams all larger than 1/8” width/dimension in any combination	1.1
	Profile with more than 4 roots, cracks, sand/silt seams all larger than 1/8” width/dimension in any combination	1.2

Determine edge moisture variation distance, e_m for both center lift and edge lift from the e_m Selection Chart, Figure 3.6, using the larger value obtained from I_m chart or α' chart.

3.6.2 Calculate γ_h

Determine γ_o using the following steps:

Step 1

Determine Mineral Classification Zone I, II, III, IV, V or VI from the Mineral Classification Chart, (See Figure 3.7). If the 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 Plasticity Index of 7, bounded by the U-Line and the A-Line, use $\gamma_o = 0.01$

Step 2

Proceed to the chart corresponding to the zone determined in Step 1 to determine γ_o. (See Figures 3.7, 3.8, 3.9, 3.10, 3.11, 3.12 and 3.13). Interpolate between γ_o lines. Beyond extreme values of the contours, use the nearest values for γ_o.

Table 3.7 - Determining the Weighted Suction-Compression Index

Layer	Depth, D (ft)	Weight Factor, F	$F \times D$	PI	LL	Zone	(%fc)	PI/(%fc)	LL/ (%fc)
1	2	3	6	35	55	II	35	1.00	1.57
2A	1	3	3	62	80	I	55	1.13	1.45
2B	2	2	4	62	80	I	55	1.13	1.45
3A	1	2	2	48	70	II	50	0.96	1.40
3B	3	1	3	48	70	II	50	0.96	1.40
Sum	9		18						

Layer	γ_o	$\gamma_h = \gamma_o \times$ (%fc)	$\gamma_{h\ swell}$	$\gamma_{h\ swell} \times F \times D$	$\gamma_{h\ shrink}$	$\gamma_{h\ shrink} \times F \times D$
1	0.17	0.060	0.063	0.379	0.056	0.336
2A	0.15	0.083	0.090	0.269	0.076	0.228
2B	0.15	0.083	0.090	0.358	0.076	0.228
3A	0.17	0.085	0.093	0.185	0.078	0.156
3B	0.17	0.085	0.093	0.278	0.078	0.234
Sum				1.469		1.259
			$(\gamma_{h\ swell})_{mod}$	0.082	$(\gamma_{h\ shrink})_{mod}$	0.070

3.7 Moisture Barriers

Vertical moisture barriers may be used to reduce the soil support parameters (e_m and y_m) provided the barriers are properly designed to virtually stop moisture migration to or from the foundation area on a permanent basis, around the entire perimeter.

The effect of a barrier on e_m and y_m may be estimated by the principles of un-saturated soil mechanics, most easily by the use of a two-dimensional moisture flow analysis computer program, such as VOLFLO[36].

A vertical barrier should extend at least 2.5 ft below adjacent ground surface to be considered as having any significant effect.

An approximation of the effect of a vertical barrier on e_m can be obtained by using Table 3.8.

3.8 Slab Shape Factor

The Shape Factor (SF)is defined in 4.5.1. If the Shape Factor (SF) exceeds 24 the designer should consider modifications to foundation foot print, strengthened foundation systems, soil treatment to reduce swell or the use of additional non-prestressed reinforcement and/or additional ribs in areas of high torsional stresses. Analysis by finite element procedures may also be used in the case of SF>24. Geotechnical approaches should reduce $y_{m\text{-center}}$ to less than 2.0 in. and $y_{m\text{-edge}}$ to less than 1.0 in. Techniques to accomplish this could include water injection, lime or chemical injection, removal and replacement with low expansive soil materials or perimeter barriers. Geotechnical analysis should also consider the reduction of e_m by the selected technique. The depth of removal and replacement with low expansive or moisture conditioned materials, or of moisture pre-conditioned soil depth may be considered as having an effect equal to a perimeter barrier of similar depth, but each treatment approach should be individually evaluated by the geotechnical engineer. When select fill or granular material is used in the removal and replacement method, extreme care needs to be taken so that an undrained "bathtub" is not created.

Table 3.8 Values of Reduced e_m for Various Perimeter Vertical Moisture Barriers

		Depth of Barrier (ft)					
		2.5	3.0	3.5	4.0	4.5	5.0
e_m (ft) (Center or Edge)	2	2.0	2.0	2.0	2.0	2.0	2.0
	3	2.0	2.0	2.0	2.0	2.0	2.0
	4	3.1	2.6	2.0	2.0	2.0	2.0
	5	4.3	4.0	2.8	2.0	2.0	2.0
	6	5.5	5.2	4.2	3.0	2.0	2.0
	7	6.5	6.3	5.5	4.5	3.2	2.0
	8	7.6	7.4	6.6	5.7	4.7	3.3
	9	8.6	8.5	7.7	6.9	6.0	4.9

The change of y_m for various barrier depths requires analysis using a computer program, such as VOLFLO[36].

region of the roots. Redirecting surface runoff channels or swales by the owner can result in improper drainage as detailed above. To minimize movements in soils, landscaping should be done on all sides adjacent to the foundation and drainage away from the foundation should be provided and maintained.

4.4.9 Irrigation

Watering should be done in a uniform, systematic manner as equally as possible on all sides to maintain the soil moisture content consistent around the perimeter of the foundation. Areas of soil that do not have ground cover may require more moisture as they are more susceptible to evaporation. Ponding or trapping of water in localized areas adjacent to the foundations can cause differential moisture levels in subsurface soils.

4.4.10 Trees

Studies have shown that trees planted within half of their mature height from the edge of the foundation have caused differential foundation movements. These will require more water in periods of extreme drought and in some cases a root injection system may be required to maintain moisture equilibrium.

4.4.11 Dry Periods

During extreme dry periods, close observations should be made around foundations to ensure that adequate watering is being provided to keep soil from separating or pulling back from the foundation.

4.5. Structural Parameters

Post-tensioned concrete foundations, designed in accordance with the PTI method, qualify as "**designed**" footings in accordance with the IBC 2003 Chapter 18, *Soils and Foundations*, and UBC 1997 Chapter 18, F*oundations and Retaining Walls*. As such, post-tensioned slabs-on-ground are exempt from UBC 1997 Section 1806.7.2. **Designed** footings include post-tensioned slabs-on-ground in accordance with this document (see IBC 2003 Section 1805.8.2 and UBC 1997 Section 1816.) The PTI method presents a specific engineered procedure to resist the effects of expansive and compressible soils, and contains a complete set of structural design provisions.

Post-tensioned foundations in areas of high seismic risk (for example, IBC Categories D, E, and F or UBC Zones 3 and 4) should conform to the appropriate seismic provisions of the governing building code. This may include IBC 2003 Sections 1805.9 and UBC 1997 Sections 1806.6 and 1809.1 through 1809.4, and ACI 318-02 Section 22.10. These sections may require the placement of mild reinforcement in addition to post-tensioned tendons. Local mild reinforcement may be required, among other reasons, to account for stress concentrations, to act as chords or collectors, or to resist local shears or bending moments induced by shear wall hold downs.

The design procedure presented herein can be used for ribbed foundations (consisting of a uniform thickness slab with ribs projecting from the bottom of the slab in both directions) and uniform thickness foundations (a solid slab with uniform thickness and ***no*** interior ribs).

4.5.1 Slab Shape

The slab plan geometry is generally fixed by functional and architectural requirements. Experience has shown that some irregular shapes cannot properly resist bending as calculated by the method of overlapping rectangles, due to torsion effects.

It is recommended that a "Shape Factor" calculation be applied to test this condition in the form of:

$$SF = \frac{\textit{Foundation Perimeter}^2}{\text{Foundation Area}} \qquad (\textbf{4-1})$$

If SF exceeds 24, the designer should consider modifications to the foundation footprint, strengthened foundation systems, soil treatment to reduce swell or the use of additional non-prestressed reinforcement and/or additional ribs in areas of high torsional stresses. . Analysis by finite element procedures may also be used in the case of SF>24.

4.5.2 Ribbed Foundations

4.5.2.1 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. Rib spacing S shall be a maximum of 15 ft. For analysis, a minimum rib spacing of 6 feet shall be used in moment and shear equations; if actual spacing is less than 6 feet, it may be used to determine section properties. Additional ribs may be required where heavy loads are applied to the foundation, as in the case of a fireplace or an interior column. When rib spacings vary, the average spacing may be used for design unless the ratio between the largest and smallest spacing exceeds 1.5. In that case, the design spacing shall be 0.85 times the largest spacing.

Corners of ribbed foundations require special consideration. Bending moments are biaxial near corners, affected by both long and short direction bending. For foundations with widely spaced ribs the line of maximum moment around a corner may not cross a rib. Additional ribs, or a diagonal rib extending from the corner to the intersection of the first orthogonal ribs, may be advisable to ensure proper performance at corners.

4.5.2.2 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 which most influences the moment capacity, shear capacity, and deflections in the ribbed foundation. Frost depth, where applicable, may be a controlling factor for determining minimum edge rib depth. The computer study used in the development of the PTI design method was based upon a uniform moment of inertia across the full width of the foundation, implying that all ribs are the same depth. However there now exists substantial successful experience with ribbed foundations in which the design was based upon more than one rib depth (for example, a deeper edge rib and shallower interior ribs). For many years a computer program available through the Institute has permitted the use of two rib depths in determining foundation section properties. Based upon that successful experience, it is permissible to use ribs of different depths in the design, provided that the ratio between the deepest and shallowest rib does not exceed 1.2. In addition, the total rib depth h shall be in no case less than 11 in., and the rib must extend **at least** 7 in. below the bottom of the slab ($h \geq t$ + 7 in.).

4.5.2.3 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 deflections (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. 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.

Excavated ribs less than 8 in. wide may be impractical due to excavation considerations. Rib widths greater than 14 in. may be used if required for bearing, however in that case a width of 14 in. shall be used in section property calculations. Excavated rib widths most commonly found in practice are 10 to 12 in.

Observations of numerous foundations built on soils with low bearing values and designed using larger bearing areas (containing a portion of the slab in addition to the rib width) have shown satisfactory performance.

For that reason, it is recommended that the applied soil bearing pressure be based upon the bearing area of the rib bottoms plus a portion of the slab equal to 16 times the slab thickness for interior ribs and 6 times the slab thickness for exterior ribs. The applied soil bearing pressure calculated in accordance with these bearing areas shall not exceed the allowable soil bearing pressure specified by the geotechnical engineer.

4.5.2.4 Rib Continuity

Ribs should be continuous between the edges of the foundation in both directions. To be considered as a continuous rib in the design rectangle the rib shall be (a) continuous or (b) overlap a parallel rib with adequate length and proximity so as to be effectively continuous or (c) be connected to a parallel rib by a perpendicular rib which transfers by torsion the bending moment in the rib.

4.5.3 Uniform Thickness Foundations

To design a uniform thickness foundation, the designer must first design a ribbed foundation for moment, shear, and stiffness, and then convert the ribbed foundation to a uniform thickness foundation using a conversion equation. The original ribbed foundation must conform to all of the moment, shear, and stiffness requirements for ribbed foundations, including the lim-

This equation was derived by relating permissible deflection criteria from previous editions to resulting parabolic curvature relationships across the appropriate slab length. The committee feels 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. Minimum stiffness required by Eqn. (*6-22*) should be determined for each direction in both swell modes. The actual foundation stiffness $E_{cr}I$ in each direction should be the larger of the two stiffnesses computed by Eqn. (*6-22*) for that direction.

The coefficient C_Δ is a function of the type of superstructure material and the swelling condition (center or edge lift). Recommended values of C_Δ for both swelling conditions and various superstructure materials are shown in Table 6.2. Smaller values of C_Δ may be used for superstructure materials listed in Table 6.2 if effective jointing details are used to minimize cracking, such as closely spaced control joints in brick or stucco walls. See 4.5.6 for a discussion of the C_Δ coefficients for prefabricated roof trusses.

6.11 Shear

6.11.1 Applied Service Load Shear:

Expected values of service shear forces in kips per foot of width of foundation and stresses in kips per square inch may be calculated from the following formulas:

6.11.1.1 Center Lift:

(a) Short Direction Shear:

$$V_S = \frac{1}{1350}\left(L^{0.19}S^{0.45}h^{0.20}P^{0.54}y_m^{0.04}e_m^{0.97}\right) \quad (6\text{-}23)$$

(b) Long Direction Shear:

$$V_L = \frac{1}{1940}\left(L^{0.09}S^{0.71}h^{0.45}P^{0.44}y_m^{0.16}e_m^{0.93}\right) \quad (6\text{-}24)$$

6.11.1.2 Edge Lift (for both directions):

$$V_L = V_S = \frac{L^{0.07}h^{0.4}P^{0.03}e_m^{0.16}y_m^{0.67}}{3S^{0.015}} \quad (6\text{-}25)$$

6.11.1.3 Applied Service Load Shear Stress, v:

(a) Ribbed Foundations:

$$v = \frac{VW}{nhb} \quad (6\text{-}26)$$

Only the rib area is considered in calculating the cross-sectional area resisting shear force in a ribbed foundation.

(b) Uniform Thickness Foundations:

$$v = \frac{V}{12H} \quad (6\text{-}27)$$

6.11.2 Compare v to v_c. If v exceeds v_c, provide shear reinforcement in accordance with the following:

$$\frac{A_v}{s} = \frac{(v - v_c)\,b}{0.4f_y} \quad (6\text{-}28)$$

Possible alternatives to shear reinforcement include:

6.11.2.1 Increasing the rib depth

6.11.2.2 Increasing the rib width

6.11.2.3 Increasing the number of beams (decrease the rib spacing)

6.12 Uniform Thickness Conversion

Once the ribbed foundation has been designed to satisfy moment, shear, and stiffness requirements, it may be converted to an equivalent uniform thickness foundation with thickness H, if desired. The conformant ribbed foundation being converted to a uniform thickness foundation need not satisfy soil bearing stress requirements based upon the bearing area specified for ribbed foundations (see 4.5.2.3), provided that soil bearing stress requirements in the uniform thickness foundation are satisfied based upon the entire plan area. Since there are methods available to comply with the cracked sections provisions (see 6.9) without changing the moment of inertia, the conformant ribbed foundation need not satisfy the cracked section provisions.

The following equation for H shall be used for the conversion:

$$H = \sqrt[3]{\frac{I}{W}} \quad (6\text{-}29)$$

Equation (**6-29**), like all equations in this document, is unit-specific. Terms must be entered with appropriate units as defined in Appendix A.1 (I in in^4, W in ft, H in in.).

After the uniform thickness conversion is complete, the designer should verify that the flexural and shear stresses of the uniform thickness foundation do not exceed the allowable stresses in 6.5 (see 4.5.3 for prestress force requirements in the uniform thickness foundation). The uniform thickness foundation should also be checked to ensure that minimum stiffness and cracked section requirements are met.

6.13 Other Applications of Design Procedure

The design procedure presented in this manual has other practical slab-on-ground applications besides construction on expansive clays, as discussed below:

6.13.1 Design of Non-Prestressed Slabs-on-Ground:

The equations for predicted moment, shear, and required stiffness are applicable for slabs reinforced with non-prestressed as well as prestressed reinforcement, and slabs designed for less than these design values are likely to have insufficient capacity to adequately resist or span over the anticipated soil movement. Once these design parameters are known, design of either type of foundation can proceed.

This design manual does not provide design procedures for non-prestressed slabs-on-ground. Non-prestressed foundations designed on the basis of cracked sections must use cracked section properties for deflection calculations, and will generally require significantly deeper ribs than those in prestressed foundations.

6.13.2 Design of Slabs Subject to Frost Heave:

Applied moments, shears and deflections due to frost heave can be approximated by substituting anticipated frost heave for expected swell of an expansive clay. The values of e_m and y_m for frost heave must be estimated from values comparable to those for expansive soils.

6.13.3 Slabs-on-Ground Constructed on Compressible Soils:

Design of slabs constructed on compressible soils can be done in a manner similar to that of the edge lift condition for slabs on expansive soils. Compressible soils are discussed in 3.2.2 and are normally assumed to have long term settlement from structural and fill loads exceeding 1.5 in. total and 0.75 in. differentially. Special design equations are necessary for this problem due to the expected *in situ* elastic property differences between compressible soils and the stiffer expansive soils. This procedure is illustrated in a design example presented in Appendix A.8 of the 2nd Edition of "Design and Construction of Post-Tensioned Slabs-on-Ground." (Also available as a technical reprint from PTI.) These equations are:

6.13.3.1 Moment:

(a) Long Direction:

$$M_{cs_L} = \left(\frac{\delta}{\Delta_{ns}}\right)^{0.5} M_{ns} \qquad (\textbf{6-30})$$

(b) Short Direction:

$$M_{cs_S} = \left(\frac{970 - h}{880}\right) M_{cs_L} \qquad (\textbf{6-31})$$

where:

$$M_{ns} = \frac{(h)^{1.35}(S)^{0.36}}{80(L)^{0.12}(P)^{0.10}} \qquad (\textbf{6-32})$$

$$\Delta_{ns} = \frac{(L)^{1.28}(S)^{0.80}}{133(h)^{0.28}(P)^{0.62}} \qquad (\textbf{6-33})$$

6.13.3.2 Differential Deflection:

$$\Delta_{cs} = \delta \exp\left(1.78 - 0.103h - 1.65x10^{-3}P + 3.95x10^{-7}P^2\right) \qquad (\textbf{6-34})$$

6.13.3.3 Shear

(a) Long Direction:

$$V_{cs_L} = \left[\frac{\delta}{\Delta_{ns}}\right]^{0.30} V_{ns} \qquad (\textbf{6-35})$$

(b) Short Direction:

$$V_{cs_s} = \left[\frac{116 - h}{94}\right] V_{cs_L} \qquad (\textbf{6-36})$$

where:

$$V_{ns} = \frac{(h)^{0.90}(PS)^{0.30}}{550(L)^{0.10}} \qquad (\textbf{6-37})$$

APPENDIX A.1

List of Symbols and Notation

A = Area of gross concrete cross-section, in^2

A'_b = Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the tendon anchor, in^2

A_b = Bearing area beneath a tendon anchor, in^2

A_{bm} = Total cross-sectional area of rib concrete, in^2

A_c = Activity ratio of clay

A_o = Coefficient in Equation (**6-13**)

A_{ps} = Area of prestressing steel, in^2

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 (**6-13**)

B_w = Assumed slab width (used in Section 6.14), in.

b = Width of an individual stiffening rib, in.

C = Constant used in Equation (**6-13**)

C_Δ = Coefficient used to establish minimum foundation stiffness (see Table 6.2)

CGC = Geometric centroid of gross concrete section

CGS = Center of gravity of prestressing force

C_p = Coefficient in Equation (**6-43**) for slab stress due to partition load - function of k_s

CR = Prestress loss due to creep of concrete, kips

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

E_c = concrete modulus of elasticity = $33w^{1.5}\sqrt{f'_c}$ psi

E_{cr} = Long-term or creep modulus of elasticity of concrete, psi. Unless more refined calculations are used, E_{cr} may be taken as 1,500,000 psi.

EI = Expansion index (see UBC 1991 Table 29-C and UBC Standards Section 29-2)

ES = Prestress loss due to elastic shortening of concrete, kips

e_p = Eccentricity of post-tensioning force (perpendicular distance between the CGS and the CGC), in.

e_1, e_2 = Void ratios corresponding to the respective overburden pressures P_1 and P_2 (see 3.6.2.1.)

e_m = Edge moisture variation distance, ft

e = Base of natural (Naperian) logarithms

F_f = Fabric factor used to modify unsaturated diffusion coefficient (α) for presence of roots, layers, fractures, and joints (See Table 3.1).

f = Applied flexural concrete stress (tension or compression), psi

f'_c = 28-day concrete compressive strength, psi

f'_{ci} = Concrete compressive strength at time of stressing tendons, psi

f_{bp} = Allowable bearing stress under tendon anchorages, psi

f_c = Allowable concrete compressive flexural stress, psi

f_{cr} = Concrete modulus of rupture, flexural tension stress which produces cracking, psi

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

f_p = Minimum average residual prestress compressive stress, psi

f_{pi} = Allowable tendon stress immediately after stressing, ksi

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

f_{pu} = Specified maximum tendon tensile stress, ksi

f_t = Allowable concrete flexural tension stress, psi

g = Moment of inertia factor

h = Total depth of stiffening rib, measured from top surface of slab to bottom of rib, in.

H = Thickness of a uniform thickness foundation, in.

I = Gross concrete moment of inertia, in^4

I_m = Thornthwaite Moisture Index

k = Depth-to-neutral axis ratio; also abbreviation for "kips"

k_s = Soil subgrade modulus, pci

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

LL = Liquid Limit, %

L_L = Long length of the design rectangle, ft

L_S = Short length of the design rectangle, ft

M_{cs} = Applied service moment in foundation on compressible soil, ft-kips/ft

M_L = Maximum applied service load moment in the long direction (causing bending stresses on the short cross-section) from either center lift or edge lift swelling condition, ft-kips/ft

M_{max} = Maximum moment in foundation under load-bearing partition, ft-kips/ft

M_{ns} = Moment occurring in the "no swell" condition, ft-kips/ft

M_S = Maximum applied service load moment in the short direction (causing bending stresses on the long cross-section) from either center lift or edge lift swelling condition, ft-kips/ft

N_T = Number of tendons

n = Number of stiffening ribs in a cross-section of width W

P = A uniform unfactored service line load (P) acting along the entire length of the perimeter stiffening ribs representing the weight of the exterior building material and that portion of the superstructure dead and live loads which frame into the exterior wall. P does not include any portion of the foundation concrete.

P_b = Point load, kips

P_e = Effective prestress force after losses due to elastic shortening, creep and shrinkage of concrete, and steel relaxation, kips

PI = Plasticity Index, %

PL = Plastic Limit, %

P_i = Prestress force immediately after stressing and anchoring tendons, kips

P_r = Effective prestress force considering subgrade friction = P_e - SG, kips

P_s = Prestress force at jacking end immediately before anchoring tendons, typically $0.8A_{ps}f_{pu}$, kips

pF = Soil suction value

P_1, P_2 = Overburden soil pressures corresponding to void ratios e_1 and e_2 (see 3.6.2.1), psi

q_{allow} = Allowable soil bearing pressure, psf

q_u = Unconfined compressive strength of the soil, psf

RE = Prestress loss due to steel relaxation, kips

r_1 = Area ratio

S = Interior stiffening rib spacing used for moment and shear equations, ft. If rib spacings vary, the average spacing may be used if the ratio between the largest and smallest spacing does not exceed 1.5. If the ratio between largest and smallest spacing exceeds 1.5, use S=0.85 x [largest spacing]. See 4.5.2.1.

s = Spacing of rib shear reinforcement, in.

SF = Shape factor = $\frac{(Foundation\ Perimeter)^2}{(Foundation\ Area)}$ where foundation perimeter is in ft and foundation area is in ft^2

S_S = Slope of suction vs. volumetric water content curve (Ref. 71).

SCF = Stress Change Factor, used in determination of y_m.

SG = Prestress loss due to subgrade friction, kips

SH = Prestress loss due to concrete shrinkage, kips

S_b = Section modulus with respect to bottom fiber, in^3

S_t = Section modulus with respect to top fiber, in^3

t = Slab thickness in a ribbed foundation, in.

UTF = Uniform thickness foundation

V = Controlling service load shear force, larger of V_S or V_L, kips/ft

V_{cs} = Maximum service load shear force in foundation on compressible soil, kips/ft

V_L = Maximum service load shear force in the long direction from either center lift or edge lift swelling condition, kips/ft

V_{ns} = Service load shear force in the "no swell" condition, kips/ft

V_S = Maximum service load shear force in the short direction from either center lift or edge lift swelling condition, kips/ft

v = Service load shear stress, psi

v_c = Allowable concrete shear stress, psi

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

W_{slab} = Foundation weight, lbs

w = Unit density of concrete, lb/ft^3

y_m = Maximum unrestrained differential soil movement or swell, in.

z = Smaller of L or 6β in direction being considered (see 4.5.7), ft

z_m = Depth below soil surface at which the suction varies by less than 0.027pF/ft (see 3.6)

$\%fc$ = Percent of fine clay, %

α = Unsaturated diffusion coefficient, a measure of moisture movement in unsaturated soils.

α' = Unsaturated diffusion coefficient modified by soil fabric factor, $\alpha' = \alpha F_f$

μ = Coefficient of friction between foundation and subgrade

δ = Expected settlement, reported by the Geotechnical Engineer, occurring in compressive soil due to the total load expressed as a uniform load, in.

β = Relative stiffness length, approximate distance from edge of foundation to point of maximum moment $\frac{1}{12}\sqrt[4]{\frac{E_{cr}I}{1,000}}$, ft

γ_o = Change of soil volume for a change in suction for 100% fine clay

γ_h = Change of soil volume for a change in suction corrected for actual % fine clay. Also referred to as matrix suction compression index.

$\gamma_{h\ corr}$ = Suction compression index corrected for coarse-grain component of the soil

$\gamma_{h\ mod}$ = Change of soil volume for a change in suction corrected for actual % fine clay weighted for layered soils

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