Summary of Changes

Addendum No. 1 to the 3rd Edition of the Design of Post-Tensioned Slabs-on-Ground

1. **Clarify the definition of an expansive soil site -- Sections 3.2.1.1 to 3.2.1.4 and 4.1 have been modified.**

There is currently a conflict in 4.1 with the statements of 3.2.1 with respect to what constitutes an expansive soil site. Further, the description of the soil tests to define an expansive site are inconsistent with standard practice such as the weighting method originally found in the BRAB report #33, which calculate the weighted Plasticity Index of a site. For clarity, the Plasticity Index of a site down to 15 feet is now to be weighted using three five foot layers. In addition the other soil tests listed under 3.2.1.2, .3, and .4 should be likewise weighted to avoid inconsistencies and prevent the highest value from being used for the full profile even if it is only found in a very thin layer, or deep in the profile.

2. **Revise how the correction of γ_h for coarse grain soil is calculated -- Section 3.6.2 has been modified.**

It has been found by analysis that the coarse grain correction gives erroneous answers when the amount retained on the #10 Sieve is less than 10%.

3. **Clarify the modeling of layered soil profiles including non-expansive layers—Section 3.6.3 modified.**

Modeling layered soil profiles lacks clarity in the manual and it is believed important to clarify the wording to limit variations of γ_h between layers to 10% or less or use a computer modeling program. Analysis has shown significantly different answers if this approach is not used. Also, guidance is offered for modeling non-expansive layers by setting the γ_o equal to a very low number.

4. **Clarify how to select soil suction profile – Section 3.6.3 modified.**

A major part of SOG design is the resolution between the use of Post-Equilibrium and Post-Construction suction profile conditions. Table 3.2 has been revised to illustrate, by shaded rows, values that are extreme for most cases and which will provide extremely conservative results causing designs to be unnecessarily heavy. Table 3.2A provides stress change factors for use in determining Ym for the post-construction case. The previous stress change factor tables only considered the post-equilibrium case. In Table 3.2A the use of a suction change of 1.5 pF is emphasized and recommended for normal design practice, while not limiting the use of other values of suction change for use in special cases or when justified by local experience

5. **Clarification of soil fabric factor – Table 3.1 – Soil Fabric Factor modified.**

The limiting of the Fabric Factor to a normally recommended value of 1.0 for certain defined climate zones and soil types is an effort to limit the compounding of conservative approaches taken by some Geotechnical Engineers, which have led to overly conservative designs.

6. **Clarifications to Shape Factor – Section 3.8 added.**

Section 4.5.1 states that if Shape Factor (SF) exceeds 24 the designer should consider modifications to the building footprint, strengthen slab systems, soil treatment to reduce swell or alternate types of foundations. The revision gives guidance on how to implement the “soil treatment to reduce swell” phrase.

7. Clarify treatment of tendon eccentricity in Uniform Thickness Foundations -- Section 4.5.3 modified and A.3.2.5.C deleted.

The 3rd Edition requires (in 6.12 last paragraph p. 47) that the flexural capacity of the converted UTF be equal to or greater than the capacity of the conformant ribbed foundation, with the actual force and tendon location used in the UTF. This means that any tendon eccentricity used in the UTF which results in conformant flexural stresses is acceptable, and no arbitrary limitation on eccentricity is required. Further, since the same end moment applied in both ribbed and uniform thickness foundations does not produce the same flexural stresses in each (because the top and bottom section moduli are not the same even with identical moments of inertia), the requirement for equal end moments is not really accomplishing the purpose of guaranteeing equivalence between ribbed and uniform thickness foundations. That equivalence is accomplished by the stress check required for the UTF in 6.12 without the necessity of an arbitrary limit on the location of tendons in the UTF.

8. Cracked Section Capacity – Sections 4.5.7 & 6.9 modified.

This provision was added in the 3rd Edition to increase the positive moment capacity of a cracked section by adding bonded reinforcement. However, the total amount of reinforcement provided (sufficient to develop a cracked section capacity equal to 90% of the uncracked section capacity) was found to result in too conservative designs based on observed performance of slabs. The required reinforcement has been revised downward to provide at least 50% of the uncracked section capacity to reflect the additional support provided by the soil after the concrete has cracked.

9. Revise the equation for allowable shear stress—Section 6.5.4 modified.

When calculated with soil support parameters derived with the 3rd Edition geotechnical procedure, shear is controlling a significantly larger percentage of foundation designs compared to the 2nd Edition. The consistent absence of shear failures in post-tensioned foundations justifies a revision and liberalization in the equation for allowable shear stress. The allowable shear equation ($v_c = 1.7\sqrt{f_c'} + 0.2 f_{pc}$) in the 2nd Edition manual was derived assuming a modulus of rupture of $7.5\sqrt{f_c'}$. The coefficients (1.7 and 0.2) assume a factor of safety of 1.67. Recent literature indicates that a modulus of rupture of $12.0\sqrt{f_c'}$ can be used to predict the flexural strength of concrete.

Using the same factor of safety of 1.67 and a modulus of rupture of $12.0\sqrt{f_c'}$, the allowable shear equation would be $v_c = 2.4\sqrt{f_c'} + 0.2 f_{pc}$.

10. Revise the equation for required stiffness – Section 6.10 and Appendix A.2 modified.

The intent of the stiffness provisions in the 3rd Edition manual was to require stiffness levels similar to the stiffness required to comply with the differential deflection provisions in the 2nd Edition manual. With the current equation ($E_{cr} I_{LorS} \geq 18{,}000 M_{LorS} L_{SorL} C_{\Delta} z_{LorS}$) in many cases, significantly more stiffness is needed to comply which results in heavier designs than what was required using the differential deflection equations.

ADDENDUM No. 1

to the

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

May 2007

with Errata approved February 7, 2008

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Addendum No. 1 revises portions of the Post-Tensioning Institute's Design of Post-Tensioned Slabs-on-Ground, 3rd Edition published December, 2005. (Revised text is denoted by vertical bars in the margins.) Each page is numbered to correspond the associated page in the 3rd Edition manual. It is intended that the Addendum pages supercede the corresponding pages in the manual. All other pages remain unchanged and can be used directly with the Addendum pages.

Additional copies of this Addendum are available upon request to PTI or are available for free download on PTI's website at www.post-tensioning.org .

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3.0 GEOTECHNICAL INVESTIGATION

3.1 Introduction

The support parameters described in this chapter are equally applicable to all ribbed or uniform thickness foundations, with non-prestressed or prestressed reinforcement, constructed on expansive, compressible or non-active soil sites. These procedures are not applicable to collapsing soils or other highly unusual local conditions for which unique procedures have been developed. One such example of an unusual local condition is the thick marine deposit of soft, unconsolidated, seawater-saturated gray clays containing vegetative remains and soft shells, found commonly around San Francisco Bay and known locally as "bay mud". These procedures are also not intended to apply to compressible soils that are improperly compacted. The values of the expansive soil parameters (ESP) e_m (see 3.6.1) and y_m (see 3.6.3) determined by this procedure, and the resulting internal forces and differential deflections, should be used for the structural design of any shallow concrete foundations regardless of the type of reinforcement. While the support parameters, internal forces and differential deflections described in this document are fully applicable to non-prestressed foundations, specific design procedures for determining reinforcement in non-prestressed foundations are not addressed herein (See 6.13.1).

3.2 Design Principles

3.2.1 Expansive Soils Sites

Sites for which expansive soil design is applicable should satisfy 3.2.1.1 through 3.2.1.3, or 3.2.1.4. Tests showing compliance with 3.2.1.1 through 3.2.1.3 are not required if the test prescribed in 3.2.1.4 is conducted. This is consistent with the expansive soil classification found in the ***International Building Code*** (IBC) 2003 Section 1802.3.2[50].

3.2.1.1 - Plasticity Index (*PI*) of 15 or greater determined in accordance with ASTM D 4318 and the procedure of Figure 3.17 utilizing three 5-ft layers, or having a 2-ft or thicker layer within the upper five feet with a *PI* of 15 or greater.

3.2.1.2 - More than 10 percent of the soil particles pass a No. 200 sieve (75 μm), determined in accordance with ASTM D 422 and the procedure of Figure 3.17 utilizing three 5-ft layers.

3.2.1.3 - More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422 and the procedure of Figure 3.17 utilizing three 5-ft layers.

3.2.1.4 - Expansion index (*EI*) greater than 20 determined in accordance with ASTM D 4829 and the procedure of Figure 3.17 utilizing three 5-ft layers.

3.2.2 Compressible Soil Sites

These are sites in which the predominant geotechnical effect could be settlement under the imposed loads of the structure or fill. A definition of a compressible site is one in which the consolidation pressure is greater than the preconsolidation pressure as found on the e-log *P* relationship developed from consolidation testing, provided that the average applied pressure taken over the entire area of the foundation is 500 psf or smaller. It should be cautioned that a site that has an apparently compressible material in the upper few feet, becoming stiffer with depth, might experience loss of edge support during periods of extended droughts, causing the need for an expansive clay analysis to be utilized as well as the compressibility equations.

The general compressible site considerations are as follows:

1. The estimated total settlement in the center of the structure, based on applied average structural loads.
2. Allowable bearing capacity to be applied at the bottom of the stiffener ribs plus a portion of the slab (see 4.5.2.3) or in the case of uniform thickness foundations over the entire foundation.
3. If the applied average pressure does not exceed the preconsolidation pressure, for a depth within 0.85 the width of the entire foundation, it is unlikely that the site is compressible.
4. The preconsolidation pressure can be estimated based on a correlation from Skempton[99]. This correlation is as follows:

$$S_u / P_o = 0.11 + 0.0037(PI) \qquad (3\text{-}1)$$

Where PI = Plasticity index in percent

Then $P_p = S/(S_u/P_o)$

Where P_p = Preconsolidation pressure

S_u = Undrained soil shear strength for a normally consolidated clay

S = Undrained soil shear strength at the depth of overburden pressure.

P_o = Overburden Pressure

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	F_f
Soil profiles contain **few** roots, layers, fractures or joints (No more than 1 per vertical foot)	1.0
Soil profiles contain **some** roots, layers, fractures or joints (2 to 4 per vertical foot)	1.3
Soil profiles contain **many** roots, layers, fractures or joints (5 or more per vertical foot)	1.4

Notes:

1. Roots, sand or silt seams or desiccation cracks should be 0.125 in. in width or greater to be considered in the vertical count.

2. A typical value for the Soil Fabric Factor should be 1.0 unless it is clear from samples and logging that the description in the table has been met.

3. A Fabric Factor of 1.3 or 1.4 is frequently used for the CH clay layers within the profile and meeting the above descriptions for Thornthwaite Indices between +20 and -20.

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.

Equation ***3-11***:

$$F = \frac{100}{1+\left(\frac{6}{100-6}\right)\left(\frac{103.0}{2.65 \text{ x } 62.4 \text{ pcf}}\right)}$$

$$F = \frac{100}{1+0.0638 \text{ x } 0.6229}$$

$$F = 96.2$$

Equation ***3-10***:

$$(\gamma_h)_{corr} = 0.076\left[\frac{100}{96.2\left(\frac{103.0}{95.0}\right)+(100-96.2)}\right]$$

$$(\gamma_h)_{corr} = 0.076 \text{ x } 0.925$$

$$(\gamma_h)_{corr} = 0.070$$

The correction of γ_h for coarse grained soil should only be used in cases where the percentage retained on the #10 Sieve is 10% or more.

3.6.3 Differential Soil Movement (y_m)

Differential soil movement should be estimated using the change in soil surface elevation at two locations separated by a distance e_m within which the differential movement will occur. An initial and a final suction profile should be used at the edge of the foundation to determine differential movement. The initial profile may be equilibrium suction or a wet or dry profile, depending on conditions that are believed to be present at time of construction.

The final suction profile at each location should be determined from controlling suction conditions at the surface. A computer analysis of the layered profile with measured or estimated suction profile envelopes may be used to yield estimates of movement for the purpose of design and analysis, and to study the effects of trees, edge barriers, flower beds, or lawn watering.

Surficial and subsurface anomalies sometimes control the values of the total soil suction at a site. Evaluation and quantification of these anomalies should be accomplished with adequate field investigation and laboratory testing.

In the absence of local observations, controlling soil suction values at the ground surface outside the foundation are recommended as follows:

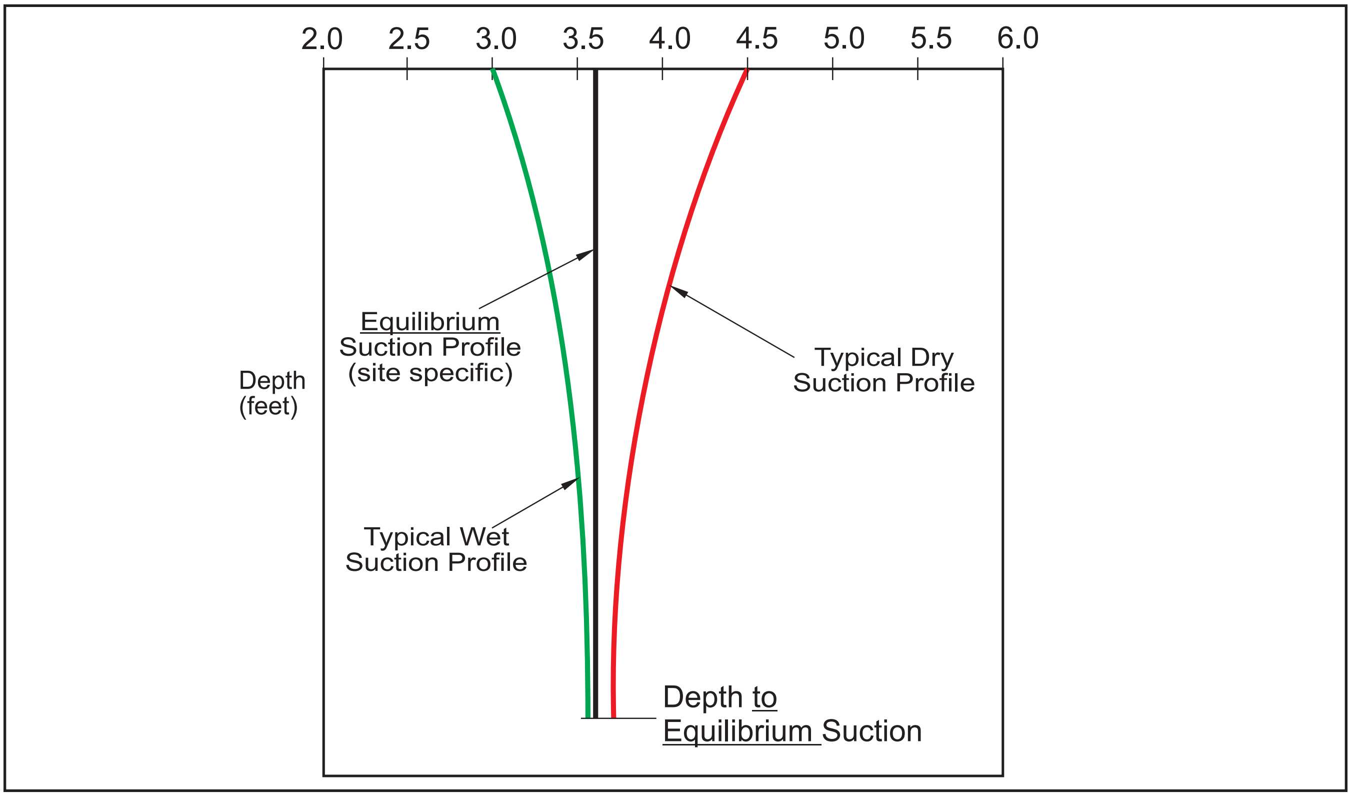
1. Wettest: 3.0 *pF* which is a typical low value for a well drained site. A 2.5 *pF* is an extreme suction value that may be used to model long term saturation conditions, and should not be used for typical design conditions.
2. Driest: 4.5 *pF* which is a typical high value to be used for normal design conditions. A value of 6.0 *pF* is an extreme upper bound representing long term sun-baked bare ground and should not be used for typical design conditions.
3. In general, typical design practice for the Post-Construction Case should use a suction variance at the ground surface of 1.5 *pF* from wettest to driest or vice-versa. This design case is recommended for geographical areas with Thornthwaite Indices between +15 and -15. The Post-Construction Case assumes swell is calculated from the extreme dry profile to the extreme wet profile, with the reverse used for shrink.
4. Geographical areas with Thornthwaite Indices drier than -15 and wetter than +15 should generally use the Post-Equilibrium Case. Unless compelling geotechnical analysis indicates otherwise, a suction profile change of 1.5 *pF* should be used with the changes between equilibrium and dry and equilibrium and wet profiles allocated per local practice. <u>In this case swell would be calculated from equilibrium to the wet profile and shrink from equilibrium to the dry profile..</u>

Controlling soil suction values below the soil surface occur at depths that are remote from the surface (z_m) and are as follows:

1. High Water Table: 2.0 *pF* at the water table unless there is an osmotic component higher than 2.0 *pF*, in which case, the measured value of suction should be used.
2. Climate-Controlled Suction: This suction may be determined by measurement at a depth below which the suction varies by less than 0.027 *pF* per ft.
3. Tree Root Zone: 4.5 *pF* under driest conditions, when the tree is near the wilting point.
4. High Osmotic Suction or Cemented Soil: These suction values must be determined by measurement. Suction at depths that are substantially different than those estimated by the Soil Suction vs Thornthwaite Moisture Index curve in Figure 3.4 indicates dissolved salts in the pore water, possible formation from deposition in a marine environment, or cementation.
5. Controlling Suction in Fill Materials:

   When foundations are to be placed on deep compacted fill materials, the geotechnical engineer who conducts the compaction quality control testing of the fill should measure the suction in each compacted layer of each distinctive soil that is used in the compacted fill beneath each foundation. Samples should be taken for suction testing at the same time that moisture density measurements are

Figure 3.16 Soil Suction Profiles



same time that moisture density measurements are made in each layer. The average of all of the suctions measured in each of the layers should be used in place of the equilibrium suction in calculating the amount of differential heave and shrinkage, y_m, to use in the design of the foundation.

A fill should be considered to be a deep fill if the calculated design heave or shrinkage y_m of the entire depth of the fill exceeds 1.5 in. It should not be necessary to take suction samples from compacted fill layers that are deeper than 20 ft below the surface of the fill.

The controlling suction in a fill that is not considered a deep fill should be the equilibrium suction as determined by the methods described above.

A typical vertical suction profile is computed by using the principles of steady state unsaturated flow which links the controlling suction values at the soil surface to the controlling suction below the surface. The principles of steady-state unsaturated flow may be found in Ref. 24.

Computer methods should be used to generate the design values of y_m for the edge lift and center lift conditions.

In the absence of computer methods, Tables 3.2, 3.3, 3.4, 3.5 and 3.6 may be used to estimate approximate values for the Stress Change Factor (*SCF*) for a given condition used in determining the approximate design value for y_m. This method should only be attempted if a typical trumpet-shaped suction profile (see Figure 3.16) can be assumed for the final suction profile and γ_h does not vary by more than 10%. Otherwise, this procedure may not be accurate or conservative. If γ_h varies by more than 10%, a computer modeling program such as VOLFLO[36] is required to accurately calculate y_m. Non-expansive layers may be modeled using γ_o equal to 0.01.

In addition, the Tables' values assume the initial suctions to be at equilibrium from depth z_m to the ground surface, then either becoming wet or dry. This limitation would not permit 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.

For active soil depths greater than 9 ft or to model post-equilibrium conditions using assumed or known soil suction profiles, the procedure requires the use of a two-dimensional analysis, which can be accomplished by using a computer solution.

Table 3.2a Stress Change Factor (SCF) for Use in Determining y_m - Post-Equilibrium Case

Measured Suction (pF) at Depth z_m	Final Controlling Suction At Surface, pF						
	2.5	2.7	3.0	3.5	4.0	4.2	4.5
2.7	+3.2	0	-4.1	-13.6	-25.7	-31.3	-40.0
3.0	+9.6	+5.1	0	-7.5	-18.2	-23.1	-31.3
3.3	+17.7	+12.1	+5.1	-2.6	-11.5	-15.8	-23.1
3.6	+27.1	+20.7	+12.1	+1.6	-5.7	-9.4	-15.8
3.9	+38.1	+30.8	+20.7	+7.3	-1.3	-4.1	-9.4
4.2	+50.4	+42.1	+30.8	+14.8	+3.2	0	-4.1
4.5	+63.6	+54.7	+42.1	+23.9	+9.6	+5.1	0

Notes:

1. Z_m = 9.0 ft.
2. Post-Equilibrium Case, which is recommended for use for areas of Thornthwaite Indices more negative than -15 and more positive than +15.
3. Shaded boxes represent extreme cases.
4. Non-typical trumpet-shaped suction envelopes or depths to Equilibrium Suction which may vary from 9 ft require use of a computer analysis.

Table 3.2b Stress Change Factor (SCF) for Use in Determining y_m- Post-Construction Case

Suction Change pF	1.3	1.4	**1.5**	1.6	1.7	1.8	1.9	2.0
Wetting (Swelling)	33.2	36.7	**40.2**	43.9	47.6	51.4	55.3	59.2
Drying (Shrinking)	-24.3	-26.7	**-29.2**	-31.7	-34.2	-36.7	-39.3	-41.9

Notes:

1. A suction change of 1.5 pF is recommended; this value has been found to produce designs which are typical and perform well in Slab-on-Ground design practice. Other values of suction change are offered for engineers to use for special cases or different local practice.
2. Z_m = 9.0 ft
3. Table 3.2b is based on Post-Construction Case which is recommended for areas of Thornthwaite Indices between -15 and +15.
4. Non-typical trumpet-shaped suction envelopes or depths to Equilibrium Suction which may vary from 9ft require use of computer analysis.

Table 3.3 Stress Change Factor (SCF) for Use in determining y_m

Measured suction (pF) at Depth z_m	Stress Change Factor							
	Controlling Surface Suction Due to Lawn Watering							
	pF - units				With 4 ft Deep Moisture Barrier pF - units			
pF	2.5	2.7	3.0	3.5	2.5	2.7	3.0	3.5
2.7	3.2	0	0	0	0.1	0	0	0
3.0	9.6	5.1	0	0	0.1	0.1	0	0
3.3	17.7	12.1	5.1	0	0.1	0.1	0.1	0
3.6	27.1	20.7	12.1	1.6	1.3	0.5	0.1	0.1
3.9	38.1	30.8	20.7	7.3	3.8	1.9	0.5	0.1
4.2	50.4	42.1	30.8	14.8	7.7	4.9	1.9	0.1
4.5	63.6	54.7	42.1	23.9	12.4	9.1	4.9	0.8

Table 3.4 Stress Change Factor (SCF) For Use in Determining y_m: Flower Bed Case (4 ft Deep Flower Bed Moisture)

Measured suction pF at Depth z_m	Stress Change Factor						
	Controlling Surface Suction Due to Flower Bed						
	pF - units			With 4 ft Deep Moisture Barrier pF - units			
pF	2.5	3.0	3.5	2.5	2.7	3.0	3.5
2.7	3.2	0	0	0	0	0	0
3.0	13.1	7.0	0	0	0	0	0
3.3	27.3	14.2	0	3.7	1.0	0	0
3.6	48.7	35.1	1.6	11.6	6.2	1.1	0
3.9	69.5	35.1	10.2	22.5	15.2	6.4	0
4.2	90.3	56.0	21.5	35.1	26.6	15.3	2.4
4.5	111.0	76.7	42.3	49.0	39.7	26.6	9.1

Table 3.5 Stress Change Factor (SCF) For Use in Determining y_m: Tree Drying Case (Without Moisture Barrier)

Depth of Tree Root Zone, ft	Stress Change Factor						
	Measured Equilibrium Suction atDepth, Z_m pF units						
	2.7	3.0	3.3	3.6	3.9	4.2	4.5
4	-79.1	-60.1	-43.2	-28.4	-15.6	-0.1	0.0
10	-169.6	-146.3	-124.9	-82.8	-42.6^{+}	$-9.7^{\neq}$	0.0
15	-244.7	-213.6	-182.5	-108.1^{*}	-42.6^{+}	$-9.7^{\neq}$	0.0
20	-333.4	-292.9	-252.5	-108.1^{*}	-42.6^{+}	$-9.7^{\neq}$	0.0

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

Notes to Tables 3.2, 3.3, 3.4, 3.5 3.6: *The positive sign indicates edge lift (swelling) and the negative sign indicates center lift (shrinkage). Measured suction at depth is the equilibrium suction.* z_m *is the depth to constant suction.*

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. This section states that if the Shape Factor (SF) exceeds 24 the designer should consider modifications to the building footprint, strengthened slab systems, soil treatment to reduce swell or alternate types of foundations. Geotechnical appoaches 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].

4.0 DESIGN COMMENTARY

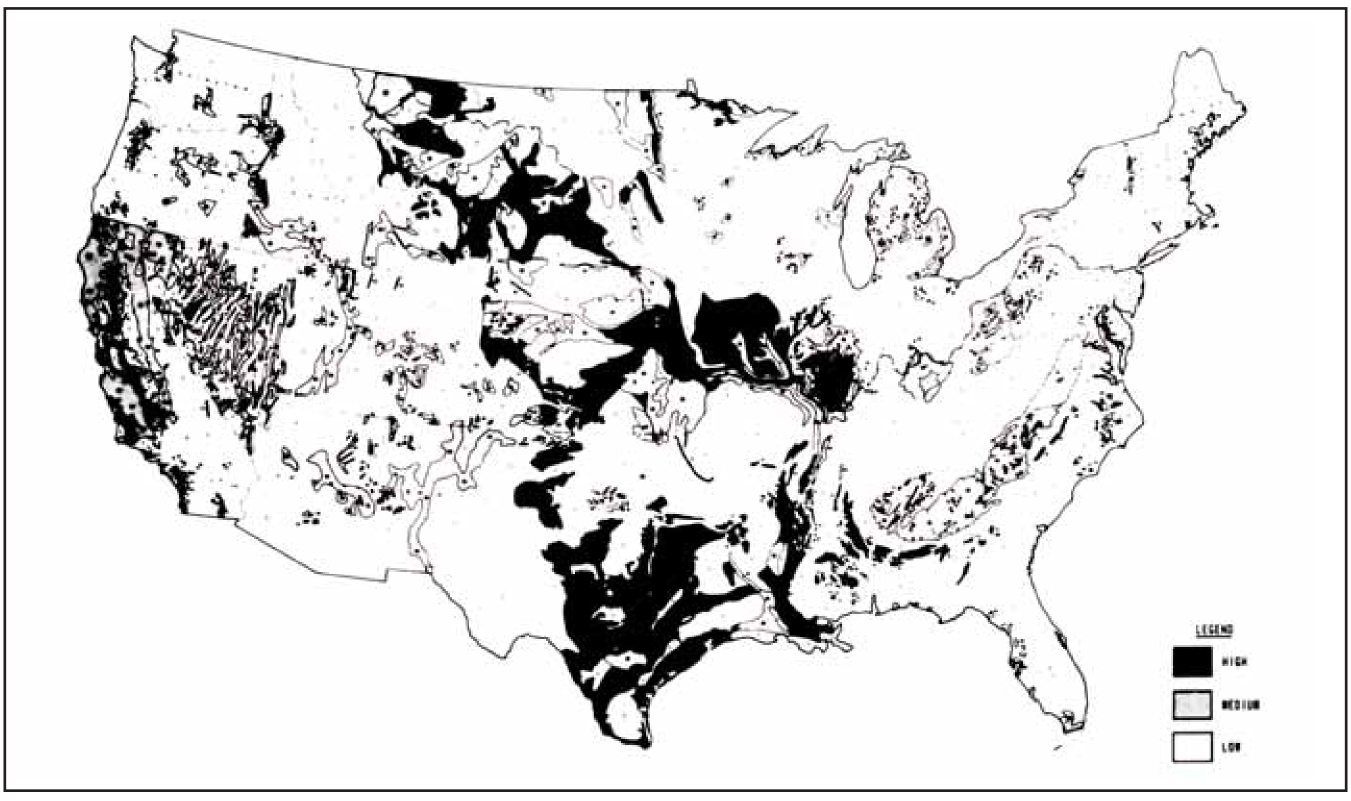
4.1 General

The design method developed herein for slabs on expansive soils is generally applicable to sites which meet the requirements of 3.2.1. The design procedure is based upon a working stress, or serviceability method. Moments, shears, and differential deflections under the action of applied service loads (including soil loading resulting from changes in climatic moisture) are predicted using equations developed from empirical data and a computer study of a plate on an elastic foundation. Concrete stresses caused by those moments and shears, acting on an assumed uncracked section, are limited to specific allowable values. Differential deflections in the slab are limited to acceptable values by providing a minimum foundation stiffness which is a function of the deformation compatibility of the superstructure.

Although the design assumes an uncracked section, it is not invalidated by the presence of shrinkage cracks, which are found to some degree in most ground-supported foundations. The effects of shrinkage and flexural cracking have been investigated, both in the original research work (see 5.2.6) and in subsequent studies[13], and in each case found to be of no significant consequence, due to the typical orientation and location of shrinkage cracks and a post-cracking increase in slab support provided by the soil. This increase in soil support also prevented the rotations necessary to develop conventional cracked section "ultimate" strengths, thus ultimate strengths are not directly considered in the design procedure. A revision is included in this edition which requires equivalent cracked and uncracked flexural capacities to account for shrinkage cracking (see 4.5.7).

A set of parameters must be known to successfully design a slab-on-ground. These include data relating to climate, soil, and structure. The design parameters discussed below are applicable to both prestressed and non-prestressed slabs-on-ground. An outline of procedures that may be used by geotechnical engineers to evaluate design properties of an expansive soil mass is presented in Chapter 3.

Figure 4.1 - Distribution of expansive soils in the United States[117]



The intent of the uniform thickness conversion is for the average compressive stress in the ribbed foundation to be maintained in the uniform thickness foundation. This will result in an increase in total prestress force in the uniform thickness foundation, since its cross-sectional area will invariably be larger than the equivalent ribbed slab. The tendons should preferably be located at the concrete centroid in the uniform thickness foundation, unless an eccentricity e_p is required to satisfy flexural stress requirements (see 6.12)

4.5.4 Loading

The loading applied to the foundation is governed by applicable building codes, the architecture of the building, framing, and the materials of construction. The design procedure developed herein assumes the following loadings on the foundation, the first two constant (built into the procedure), the third variable and determined by the designer:

4.5.4.1 Uniform Live Load

A uniform 40 psf live load applied over the entire plan area of the foundation. This is the live load applied directly on the first floor slab and does not include any live load from framed floors above the first floor.

4.5.4.2 Uniform Dead Load

A uniform 65 psf dead load applied over the entire plan area of the foundation, representing the weight of an assumed 4-in. slab plus 15 psf for partitions and other interior dead loads applied directly on the first floor slab, not including any dead loads from framed floors above the first floor.

4.5.4.3 Edge Load

A uniform unfactored service line load P acting along the entire length of the perimeter 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 from framed floors above the first floor slab and the roof. P does not include any portion of the foundation concrete.

The perimeter line loading P includes both dead load and live load. This definition of P applies for both swell modes, center lift mode and edge lift mode. In the edge lift swell mode, designers are permitted, however, to use dead load and sustained (or true long term) live load, or to use dead load only, whichever they judge to be the more appropriate.

The actual ***perimeter*** line loadings P used to develop this method ranged between 600 and 1,500 plf. Based on the past two decades of PTI method application to multi-story buildings (e.g., two to four-story wood-framed buildings) with perimeter loads exceeding 1,500 plf, the PTI method will yield reasonable results for perimeter loads in excess of 1,500 plf. Slabs designed by the PTI method with perimeter loads up to 2,500 plf have had successful performance. Engineering judgment, however, should be used for perimeter loads exceeding 1,500 plf. This procedure does not apply for slabs with ***interior*** uniform loads substantially in excess of those described above.

Unusually heavy concentrated loads, such as fireplaces, post loads, or interior bearing walls, should be evaluated on an individual basis. A formula for calculation of concrete flexural tensile stresses beneath concentrated loads is presented in 6.14. If the slab stresses produced by concentrated loads exceed those permissible, the loads should be framed to adjacent ribs in ribbed foundations, or a footing should be placed below them in uniform thickness foundations.

The structural engineer must carefully evaluate the assumption of uniform perimeter line loading as it applies to the modeling of the specific foundation under design. Actual framing can produce loads substantially different from this assumption. For example, in a rectangular building framed in the short direction, the perimeter load on the short edges will be very small (only the weight of the walls themselves) while the long edges will carry substantially *all* of the superstructure load (half on each side). In this case the assumption of uniform perimeter loading would not accurately represent the moments produced by the actual loading. In any case, the largest load intensity occurring anywhere on the perimeter should be used for design. When P varies significantly around the slab perimeter, and the ratio of largest to smallest exceeds 1.25, the largest value should be used for center lift design and the smallest value should be used for edge lift design.

4.5.5 Allowable Shear Stress

The equation for allowable shear stress (Eq. ***6-7*** in 6.5.4)) was revised in the 2nd Edition to reflect both concrete strength ***and*** prestress compression. In developing this equation, the Committee researched the relationship between the vertical shear stress and the principal tension stress, documented recommended values which have been used for the permissible principal tension stress, and the relationship of these values to current structural engineering practice for permissible vertical shear stresses.

Applied shear stress is based upon the area of the ribs only, excluding the portion of the slab outside the width of the rib. This is consistent with standard structural engineering practice for shear resistance in flanged rib sections.

4.5.6 Stiffness

Table 6.2 recommends values of C_{Δ} to determine the minimum required foundation stiffness for a wide range of superstructure materials, based upon their ability to withstand deformation. Table 6.2 recommends large C_{Δ} values for prefabricated roof trusses, resulting in large required stiffness.

In the past, significant problems (drywall cracking and wall/ceiling joint separations) have occurred in residential wood-framed structures with prefabricated roof trusses when the trusses are rigidly attached to non-bearing partitions between the truss supports. In that case, even a small relative vertical movement between the two ends of the trusses can cause unsightly gypsum wall-board cracking as a result of wall-ceiling joint separations. The large values of C_{Δ} in Table 6.2 are a warning signal to designers that this condition exists and should be mitigated.

As a preferable alternative to designing for the large C_{Δ} values for prefabricated roof trusses, joinery details (such as a preformed metal clip) can be provided between the trusses and the intersecting non-bearing partitions which permit relative vertical movement without inducing stresses into the partitions, while providing the required lateral bracing. In that case, a smaller C_{Δ} value may be used based upon the appropriate material listed in Table 6.2.

Smaller values of C_{Δ} may also be used for other superstructure materials listed in Table 6.2 if effective jointing details are used to minimize cracking, such as closely spaced control joints for brick or stucco.

4.5.7 Cracked Section Capacity

An examination of Figures 5.4 and 5.5 shows that additional post-cracking soil support can provide adequate strength to resist all applied design loads. In the edge lift swell mode (Figure 5.5) the moment/deflection curve is almost unaffected by slab cracking. In the center lift swell mode (Figure 5.4) there is a sudden "unloading" at first cracking as the cross-section rotates, but as the increased soil support is mobilized the load increases to and well beyond the level at first cracking. To account for these rotational effects, minimize crack widths, and provide ductility and equivalent cracked and uncracked section behavior, it is recommended that the total amount of reinforcement provided, both prestressed and non-prestressed, be sufficient to develop a cracked section capacity equal to 50% of the uncracked section capacity.

gle that most reasonably represents the majority of the actual foundation plan. Long narrow rectangles may not appropriately model the overall foundation and generally should not govern the design.

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:

$$SF = \frac{Foundation\ Perimeter^2}{\text{Foundation Area}}$$

If 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 $SF > 24$.

6.4 Trial Section Assumptions

Select C_Δ for edge and center lift from Table 6.2. Assume an initial rib spacing S and determine an initial rib depth as the deeper value of h from the following two equations:

6.4.1 - Center Lift

$$h = \left(\frac{(y_m L)^{0.205} S^{1.059} P^{0.523} (e_m)^{1.296} C_\Delta}{4560(z)} \right)^{0.824} \qquad \textbf{(6-1)}$$

6.4.2 - Edge Lift

$$h = \left(\frac{L^{0.35} S^{0.86} (e_m)^{0.74} (y_m)^{0.76} C_\Delta}{191(P)^{0.01}(z)} \right)^{1.176} \qquad \textbf{(6-2)}$$

If different rib depths are used in the analysis (such as a deeper edge rib) the ratio between the depth of the deepest and shallowest rib shall not exceed 1.2 (see 4.5.2.2).

6.4.2

Determine Section Properties: The moment of inertia, section modulus, and cross-sectional area of the slabs and beams, and eccentricity of the prestressing force may be calculated for the trial rib depth determined above in accordance with normal structural engineering procedures. These procedures are illustrated in the design examples presented in Appendices A.3, A.4, and A.5.

6.5 Allowable Stresses

The following allowable stresses are recommended:

6.5.1

Allowable Concrete Flexural Tensile Stress:

$$f_t = 6\sqrt{f_c'} \qquad \textbf{(6-3)}$$

6.5.2

Allowable Concrete Flexural Compressive Stress:

$$f_c = 0.45 f_c' \qquad \textbf{(6-4)}$$

6.5.3

Allowable Concrete Bearing Stress at Anchorages:

6.5.3.1

At Service Load:

$$f_{bp} = 0.6 f_c' \sqrt{\frac{A_b'}{A_b}} \le f_c' \qquad \textbf{(6-5)}$$

6.5.3.2

At Transfer:

$$f_{bp} = 0.8 f_{ci}' \sqrt{\frac{A_b'}{A_b} - 0.2} \le 1.25 f_{ci}' \qquad \textbf{(6-6)}$$

6.5.4

Allowable Concrete Shear Stress:

$$v_c = 2.4\sqrt{f_c'} + 0.2 f_p \qquad \textbf{(6-7)}$$

6.8.1.2 Short Direction

For $L_L/L_S \geq 1.1$:

$$M_s = \left(\frac{58 + e_m}{60}\right) M_L \quad \textit{(6-16)}$$

For $L_L/L_S < 1.1$:

$$M_S = M_L \quad \textit{(6-17)}$$

6.8.2 Edge Lift Moment

6.8.2.1 Long Direction

$$M_L = \frac{S^{0.1}(he_m)^{0.78}(y_m)^{0.66}}{7.2L^{0.0065}P^{0.04}} \quad \textit{(6-18)}$$

6.8.2.2 Short Direction

For $L_L/L_S \geq 1.1$:

$$M_s = h^{0.35}\left[\frac{19 + e_m}{57.75}\right] M_L \quad \textit{(6-19)}$$

For $L_L/L_S < 1.1$:

$$M_S = M_L \quad \textit{(6-20)}$$

Concrete flexural stresses produced by the applied service moments can be calculated with the following equation:

$$f = \frac{P_r}{A} \pm \frac{M_{L,S}}{S_{t,b}} \pm \frac{P_r e_p}{S_{t,b}} \quad \textit{(6-21)}$$

The applied concrete flexural stresses f should be limited to f_t in tension and f_c in compression.

The design method is based upon 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, etc.) prevent rib continuity, the designer must provide equivalent rib continuity using rational engineering approaches. To be considered continuous, ribs should 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.

6.9 Cracked Section Considerations

For design purposes the concrete flexural tensile stress is limited to $6\sqrt{f'_c}$. Since the modulus of rupture of concrete is commonly taken as $7.5\sqrt{f'_c}$ slabs designed with this method will theoretically have no ***flexural*** cracking. Some cracking is anticipated and inevitable in post-tensioned slabs on ground, as in elevated post-tensioned concrete members. Nonetheless, the limitation of flexural tensile stresses to a value less than the modulus of rupture justifies the use of the gross concrete cross-section for calculating all section properties. This is consistent with standard practices in elevated post-tensioned concrete members. Refer to 4.5.7 and 5.2.5 for additional discussion on the effects of cracking in post-tensioned ground-supported foundations.

Sufficient reinforcement, prestressed or non-prestressed, shall be provided to develop $0.5M_L$ and $0.5M_S$ for both swell modes, using conventional cracked section flexural strength methods, with a ϕ factor of 1.0. The tensile force in the prestressed reinforcement shall be taken as P_e and tensile force in non-prestressed reinforcement shall be taken as $A_s f_y$. Prestressed reinforcement located on the compression side of the section may be ignored in calculating the cracked section capacity. Non-prestressed reinforcement, if required, shall be placed perpendicular to the perimeter of the foundation, starting with a minimum concrete cover from the foundation edge, and extending inward with a minimum length of 2β. This recommendation is intended to ensure that flexural capacities at shrinkage cracks are equivalent to those at uncracked sections, and to limit shrinkage crack widths at critical sections.

6.10 Stiffness

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

$$E_{cr}\, I_{L \text{ or } S} \geq 12{,}000\, M_{L \text{ or } S}\, L_{S \text{ or } L}\, C_{\Delta}\, z_{L \text{ or } S} \quad \textit{(6-22)}$$

Allowable Concrete Bearing Stress at Anchorages:

At Service Load:

$$f_{bp} = 0.6f'_c\sqrt{\frac{A'_b}{A_b}} \leq f'_c \qquad (6\text{-}5)$$

At Transfer:

$$f_{bp} = 0.8f'_{ci}\sqrt{\frac{A'_b}{A_b} - 0.2} \leq 1.25f'_{ci} \qquad (6\text{-}6)$$

Allowable Concrete Shear Stress:

$$v_c = 2.4\sqrt{f'_c} + 0.2f_p \qquad (6\text{-}7)$$

Allowable Stresses in Prestressing Steel:

Allowable stress due to tendon jacking force:

$$f_{pj} = 0.8f_{pu} \leq 0.94f_{py} \qquad (6\text{-}8)$$

Allowable stress immediately after prestress transfer

$$f_{pi} = 0.70f_{pu} \qquad (6\text{-}9)$$

The effective prestress force P_e is:

$$P_e = P_i - ES - CR - SH - RE \qquad (6\text{-}10)$$

where, in lieu of a more detailed analysis:

$$P_i = \frac{P_S}{1 + 0.002L} \qquad (6\text{-}11)$$

Subgrade Friction

SG can be conservatively taken as:

$$SG = \frac{W_{slab}}{2000}\mu \qquad (6\text{-}12a)$$

The resultant minimum prestress force acting on the concrete cross section is

$$P_r = P_e - SG \qquad (6\text{-}12b)$$

Center Lift Moment

Long Direction

$$M_L = A_o\left[B(e_m)^{1.238} + C\right] \qquad (6\text{-}13)$$

where:

$$A_o = \frac{1}{727}\left[(L)^{0.013}(S)^{0.306}(h)^{0.688}(P)^{0.534}(y_m)^{0.193}\right] \qquad (6\text{-}14)$$

and for:

$$0 \leq e_m \leq 5 \qquad B = 1, C = 0 \qquad (6\text{-}15a)$$

$$5 < e_m \qquad B = \left(\frac{y_m - 1}{3}\right) \leq 1.0 \qquad (6\text{-}15b)$$

$$C = \left[8 - \frac{P - 613}{255}\right]\left[\frac{4 - y_m}{3}\right] \geq 0 \qquad (6\text{-}15c)$$

Short Direction

For $L_L/L_S \geq 1.1$:

$$M_s = \left(\frac{58 + e_m}{60}\right)M_L \qquad (6\text{-}16)$$

For $L_L/L_S < 1.1$:

$$M_S = M_L \qquad (6\text{-}17)$$

Edge Lift Moment

Long Direction

$$M_L = \frac{S^{0.1}(he_m)^{0.78}(y_m)^{0.66}}{7.2L^{0.0065}P^{0.04}} \qquad (6\text{-}18)$$

Short Direction

For $L_L/L_S \geq 1.1$:

$$M_S = h^{0.35}\left[\frac{19 + e_m}{57.75}\right]M_L \qquad (6\text{-}19)$$

For $L_L/L_S < 1.1$:

$$M_S = M_L \qquad (6\text{-}20)$$

Concrete flexural stresses produced by the applied service moments can be calculated with the following equation:

$$f = \frac{P_r}{A} \pm \frac{M_{L,S}}{S_{t,b}} \pm \frac{P_r e_p}{S_{t,b}} \qquad (6\text{-}21)$$

Stiffness

Minimum foundation stiffness:

$$E_{cr}\, I_{L\text{ or }S} \geq 12{,}000\, M_{L\text{ or }S}\, L_{S\text{ or }L}\, C_{\Delta}\, z_{L\text{ or }S} \qquad (6\text{-}22)$$

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