

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 Y_m 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 ($v_c = 1.7\sqrt{f'_c} + 0.2f_{pc}$) 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 $v_c = 2.4\sqrt{f'_c} + 0.2f_{pc}$.

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 ($E_{cr}I_{LorS} \geq 18,000M_{LorS}L_{SorL}C_{\Delta}Z_{LorS}$) in many cases, significantly more stiffness is needed to comply which results in heavier designs than what was required using the differential deflection equations.

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to the

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

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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 γ_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 1802.3.2⁵⁰.

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 μ m), 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 Skempton⁹⁹. This correlation is as follows:

$$S_u / P_o = 0.11 + 0.0037(PI) \quad (3-1)$$

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 (%-2 μ) expressed as a percentage of the total sample.
- Percent fine clay, $\%fC = \left(\frac{\% - 2\mu}{\% - \#200} \right) \times 100$

For example: $45\% / 80\% = 0.56 \times 100$

$= 56$, report as 56%

Calculate the unsaturated diffusion coefficient, α

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

Where:

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

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 $\gamma_{h \text{ swell}}$ and another coefficient, α' , is calculated for $\gamma_{h \text{ shrink}}$. 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 $\gamma_{h \text{ mod}}$ 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 .