COMMENTS ON THE POST-TENSIONING INSTITUTE'S "DESIGN OF POST-TENSIONED SLABS-ON-GROUND" 3RD EDITION

by

The Structural Committee

of

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PREFACE

This document was written by the Structural Committee and has been peer reviewed by the Foundation Performance Association (FPA). This document is published as FPA-SC-14 Revision 0 and is made freely available to the public at www.foundationperformance.org so all may have access to the information. To ensure this document remains as current as possible, it may be periodically updated under the same document number but with higher revision numbers such as 1, 2, etc.

The Structural Committee is a permanent committee of the Foundation Performance Association. At the time of writing this document, Ron Kelm, P.E., chaired the Structural Committee and 20 to 25 members were active on the committee. The committee sanctioned this paper March 2004 and formed a subcommittee to write the document. The subcommittee chair and members are listed on the cover sheet of this document.

Suggestions for improvement of this document may be directed to the current chair of the Structural Committee. If sufficient comments are received to warrant a revision, the committee will form a new subcommittee to revise this document. If the revised document successfully passes FPA peer review, it will be published on the FPA website and the previous revision will be removed.

This document was written specifically for use in designs based upon the Post-Tensioning Institute (PTI) Design Procedure for designing post-tensioned slabs-on-ground. The intended audiences for the use of this document are members of PTI's Slab-on-Ground Committee as well as engineers and others involved in the design of foundations for residential and other low-rise structures.

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 OVERVIEW

The objective of this document is to provide constructive comments to the Post-Tensioning Institute's (PTI's) Slab-on-Ground Committee and to inform geotechnical and foundation design engineers of the Foundation Performance Association's (FPA's) concerns regarding the Post-Tensioning Institute's "Design of Post-Tensioned Slabs-on-Ground", (PTI Design Procedure) 3rd Edition [13].

The PTI Design Procedure is used to design a slab-on grade foundation that will respond elastically to forces the foundation is exposed to including those that are the consequence of soil pressure and soil deformation. This procedure is based on empirical data, computer generated curve fits, the geotechnical engineer's computation of y_m and e_m values and other variables based on site-specific geotechnical parameters. Therefore, the geotechnical and foundation design engineers should be made aware of assumptions and limitations associated with the use of this procedure.

This paper is intended to encourage refinement of the procedure by indicating areas where additional research, clarification, and / or corrections may result in better geotechnical and structural designs. This paper addresses the PTI Design Procedure, 3rd Edition. The Foundation Performance Association's document FPA-SC-05 addresses the PTI Design Procedure, 2nd Edition [12]. The FPA refers to both editions in this paper and describes the version under discussion as the PTI Design Procedure, 2nd Edition or PTI Design Procedure, 3rd Edition.

The FPA's recommendations to the PTI Committee are included in italics at the conclusion of each section.

1.2 BACKGROUND INFORMATION

Experience has shown that foundations designed using the PTI Design Procedure usually perform well, but problems still may occur due to inadequate geotechnical information, improper maintenance, construction not in conformance to the design plans, poor foundation design, and / or design errors in the application of the PTI Design Procedure. However, there are instances where a properly designed, constructed, and maintained slab-on-ground foundation may experience performance problems.

The PTI Design Procedure was initially developed in the 1970s and has established itself as a "standard" design procedure for slab-on-ground design for low-rise and residential structures. This procedure was developed to provide a simplified design procedure to produce a product that performs with a high degree of success. The PTI Design Procedure, 3rd Edition, is the latest revision of the design procedure that will provide the design basis for a foundation that performs according to expectations. Although the PTI Design Procedure has established itself as a "standard" procedure and has been adopted into most national building codes, it is not a guarantee the procedure represents a method that is good for all designs. There are many places where the PTI Design Procedure, from a geotechnical engineering perspective, is not a reliable design and should not be used.

While well established, the PTI Design Procedure is a "work in progress" as is every other design method and building code used by the engineering profession. Therefore, foundation design and geotechnical engineers should be aware that there may be areas in any slab-on-ground design procedure that are based on imperfect data and analytical methods.

2.0 CONSIDERATIONS FOR THE REQUIRED GEOTECHNICAL INFORMATION

The basis of every foundation design should include geotechnical data. Such data are usually obtained through methods such as long-term weather data interpretations (climate), site reconnaissance, soil borings and testing of the soil samples obtained as part of the borings. The climatic and geotechnical information together can determine the design life of the structure. Local geotechnical conditions and local building practice sometimes dictate the parameters of the geotechnical investigation, such as minimum soil boring depth, sampling frequency, and field and laboratory tests. For example, Foundation Performance Association's Document No. FPA-SC-04, "Recommended Practice for Geotechnical Explorations and Reports" provides minimum geotechnical guidelines for Southeast Texas.

The geotechnical investigation should address soil compressibility, edge moisture variations and soil bearing capacity, among other parameters. The PTI Design Procedure, 3rd Edition, provides standards for minimum field investigation programs, laboratory testing programs, geotechnical report contents, site characterization recommendations and foundation design information recommendations.

2.1 THE DESIGN LIFE OF A RESIDENTIAL FOUNDATION

Primary geotechnical parameters for a foundation design, based on PTI Design Procedure, 3rd Edition, which could influence the design life of a residential foundation, are:

- Soil's bearing capacity (Q_{allow}),
- Maximum differential soil movement (y_m),
- Distances for edge moisture variation (e_m),
- Unsaturated diffusion coefficient, (α), and
- Thornthwaite moisture index, (I_m).

Except in the case of collapsible soils, the clay soil's bearing capacity is typically not as sensitive to moisture changes in the soil as are the soil movement parameters, e_m and y_m. These parameters could be used to determine the slab's expected lifetime, free of distress. Common manifestations of distress may include slab cracks due to slab movement, excessive deflection and / or tilt of the foundation.

These soil movement parameters are given for both an edge lift condition and a center lift condition. For an accurate specification of these two parameters, laboratory tests such as soil suction, free swell and hydrometer testing should be used. Many geotechnical engineers have used shortcuts to estimate these parameters. Frequently observed shortcuts include the absence of laboratory tests for compressive strength, suction, swell, shrinkage, sieve and hydrometer, all of which may cause e_m and y_m specifications to be largely judgmental and based on "experience". Systematically omitting the tests specified by the PTI may lead to erroneous values of e_m and y_m, which are crucial in the design and thus the design life of the structure. The use of proper procedures in the geotechnical site investigations and data interpretations help ensure a proper design basis.

The design life of a structure is not known to have been considered in creating the PTI Design Procedure, 3rd Edition. Environmental data and structural performance data have recently become available to address the issue of structural longevity. Dr. Robert Lytton of Texas A&M University introduced the derivation of the design life of a residential structure**.** His work resulted in a simple diagram that he presented in 1999 as Paper #16 at the Foundation Performance Association's "Seminar on Design of Foundations on Expansive Soils" [10]. His derivations are based on the number of years of data used to derive the Thornthwaite Index. The Thornthwaite Index, which provides a climate classification, is based on precipitation, temperature and potential evapotranspiration. This index describes the balance between evaporation and precipitation in a particular area. Klik [9] defines the Thornthwaite Index as the Precipitation Effectiveness Index of Thorthwaite, PE, by month, as:

$$PE = 3.16\ \Sigma\ [Pi / (1.8\ Ti + 22)]^{10/9}$$

where: Pi is monthly precipitation in mm, and
Ti is average monthly air temperature in degrees Centigrade.

In PTI Design Procedure, 3rd Edition, Figure 3.2, one can obtain a value of Thornthwaite Index for U.S. locations printed on maps based on a 20-year average value. Morin [11] developed the concept further for periods shorter than one day. From the definition one will easily recognize that it makes a difference what period is chosen to compute the Thornthwaite index. In Australasian climatic studies by Houghton and Styles [8] variations from –60 to +150 to the Thornthwaite index computed on a monthly basis are reported. Because of the natural variability in the local and annual climate it makes a significant difference over what period this index is computed.

It is well known that foundations placed during dry spells are more likely to develop foundation distress during wet periods. Soil beneath foundations responds to short-term changes in the weather that could be described by a one or two month average. Therefore, a 20-year average Thornthwaite Index is inadequate for design work. It would be better to use a statistical interpretation of the Thornthwaite Index computed for each one or two month period over the longest data records available in order to derive the design value for the Thornthwaite Index. Therefore, a more accurate design basis for the design life of a residence may be derived.

After the design life of a residence has been decided, the proper design values for e_m can be derived based on the design value for the Thornthwaite Index. Such interpretations are not available today.

Two interrelated questions remain:

1) *What is the design life of a house?* The applicable design codes should establish a design life in order to define an appropriate Thornthwaite Index. This is a complex question of which the answer has large economic consequences.

2) *How should the Thornthwaite Index be computed?* From a 20-year average or from statistical interpretation of the Thornthwaite Index computed for each one or two month period over the longest data records available?

The Thornthwaite Index is calculated in the PTI procedure for climatic conditions in geographic areas. However in arid areas of the Western United States irrigation is added to sites during development. Thus, consideration of climatic precipitation only for determination of the Thornthwaite Index for post-construction design is not conservative. In arid regions, the Precipitation Effectiveness Index of Thorthwaite, PE, will be significantly higher than that presented in the PTI procedure due to an increase of the Pi from irrigation, and a decrease of Ti from post-development local climate effects.

No data are available to aid the engineer in developing a foundation design exceeding a particular design life. The FPA concludes that the technology to design for a 30, 40, 50 and 100-year life would be beneficial to have as an option.

The FPA recommends that the PTI Design Procedure specify the basic design Thornthwaite Index for certain design lives to a probabilistic basis. For design purposes, the FPA also recommends the specification of a value of the Thornthwaite Index that has a certain probability of exceedance in a given period.

2.2 MODULUS OF SUBGRADE REACTION

In PTI Design Procedure, 3rd Edition, Section 6.14, Table 6.2 and PTI Design Procedure, 2nd Edition, Section 6.14, Table 6.1 the variables k_s (Soil Subgrade Modulus in pci, which relates to the stiffness of the soil) and E_s (Modulus of Elasticity of the soil in psi, counterpart to k_s) are used, however, PTI Design Procedure, 3rd Edition, gives no guidance on how to determine these values from field data. The variables β and β_1 in turn are a function of k_s (or E_s) and other variables that are considered constant for a given design. Because the value of β (or β_1) is used to simulate the soil stiffness in Finite Element type analyses, it follows from the design formulas in PTI Design Procedure, 3rd Edition, that the value of β (or β_1) has a major influence on the design moment, shear and deflection of the slab,

When Dr. Kent Wray developed his solution equations (that became the basis for PTI Design Procedures) he assumed a value of k_s to simulate the soil support in his Finite Element procedures. Dr. Wray's thesis [24, page 95 and 107] used a constant value of 5 pci (k_s) and a related, constant value for the modulus of elasticity for soil of 1000 psi (E_s). Das [7] defines k_s

as the spring constant for a Winkler spring, *however k_s is not constant for a given location*; it depends upon the size, depth, thickness of layers, Poisson Coefficient of soil, and size of the surface area considered, etc. Dr. Wray quotes a reference by Professor Lytton that suggests a general range of 1 to 10 pci for k_s. The FPA has also seen published estimates of k_s in reference books, such as Ringo and Anderson [18] and Das [7], but not usually in residential geotechnical site reports. A variation in k_s from 1 to 1000 pci appears to be common for foundation design. Telephone interviews with Houston geotechnical consultants resulted in a recommended k_s between 25 and 75 pci.

Given the above variations, the FPA made a sensitivity analysis of the influence of k_s on general foundation design parameters. The "SAFE" Finite Element program [19] was used to analyze a simple concrete residential ribbed slab in center lift (e_m= 9 ft) with standard values for parameters. The Finite Element model the FPA used is depicted in Figure 2.2-1.

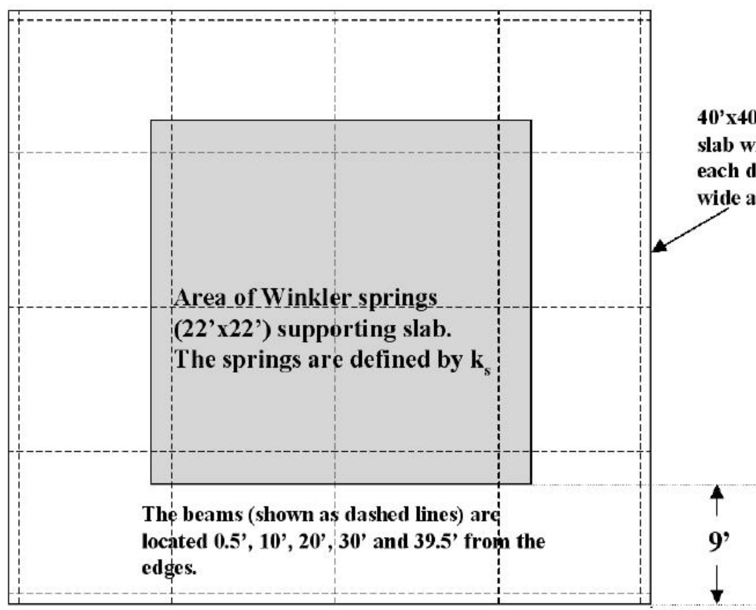

Figure 2.2-1 Model of foundation slab used for analysis of the sensitivity of the design parameters for k_s for center lift conditions.

The 40' by 40' by 4" ribbed slab has 5 ribs measuring 12" wide by 24" deep (measured below the slab) in each direction. The beams (shown as dashed lines in the figure) are located 0.5', 10', 20', 30' and 39.5' from the edges. The slab is loaded by its self-weight plus a live load of 40 lb/ft^2. This model is supported in center lift by Winkler springs in a square area of 22' by 22', corresponding to an e_m of 9' all around the slab. The maximum dimension for the finite elements was chosen as 1'. No benefit was found in using smaller elements. The stiffness of the Winkler Springs represented by the Modulus of Subgrade Reaction (k_s) was chosen as 1, 5, 10, 25, 50 and 100 pci and an analysis was run for each case.

The effect of post-tensioning was not modeled for this analysis. In the computed results reported below, lift-off (a separation of the foundation from the supporting soils) of the slab from the soils near the center of the slab was considered and iterative solutions were separately derived. Lift-off at the center of the slab was found for values of k_s ranging from 10-25 pci.

For a k_s of 25 pci a 3-D view of the deflected shape of the slab and the maximum moment is depicted in Figure 2.2-2. Because no lift-off occurred in the center, there was full soil contact for this value of k_s.

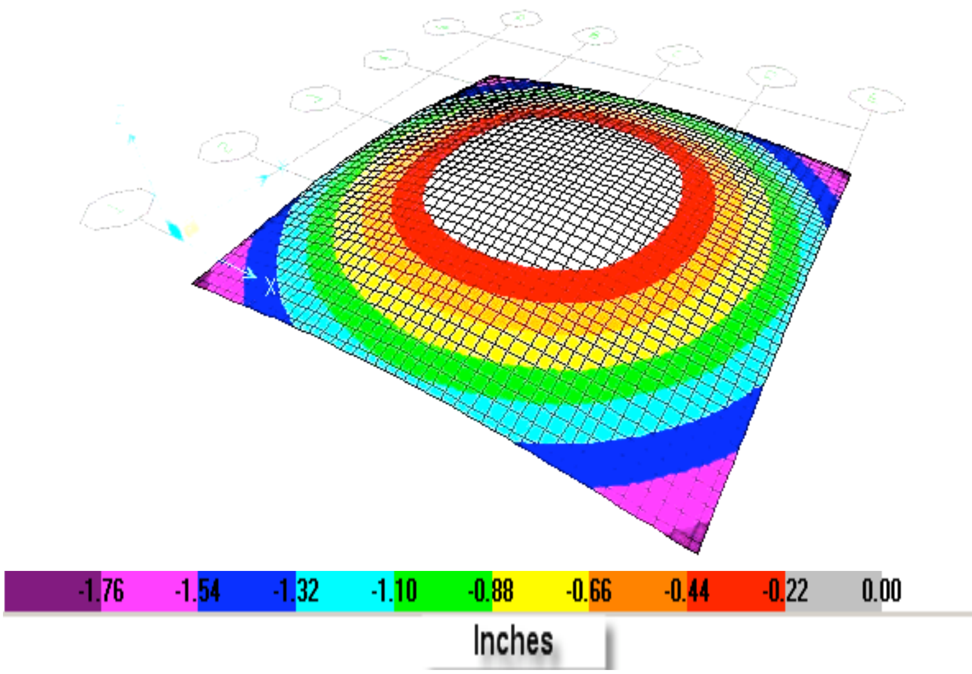

Figure 2.2-2 3-D View of typical deflected slab shape for k_s of 25 pci (Center Lift).

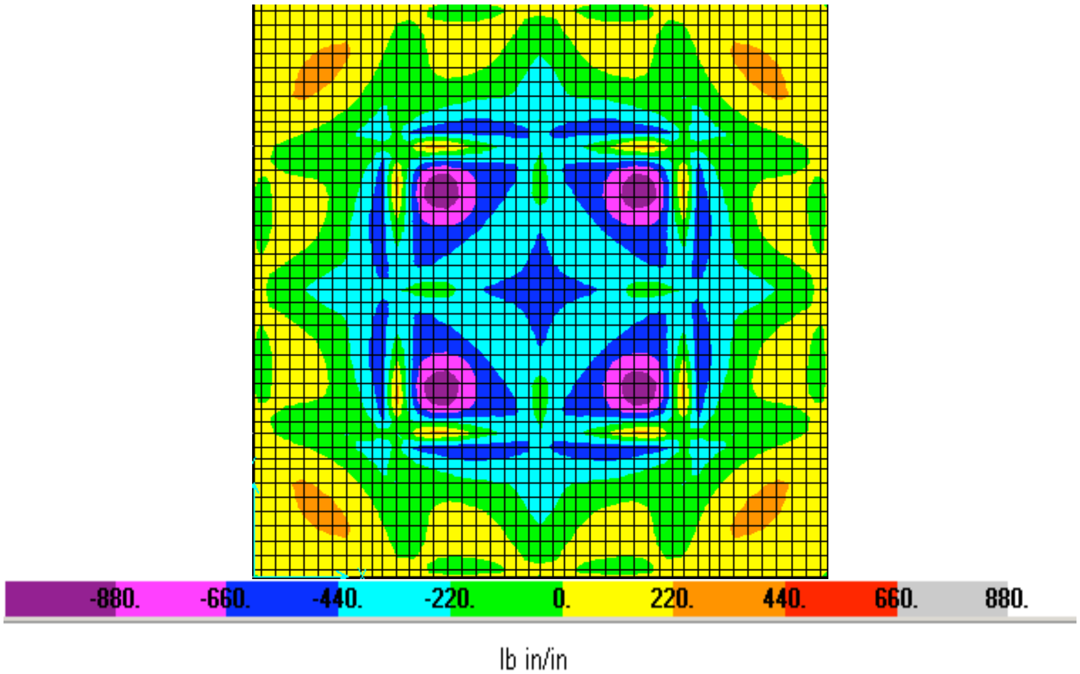

Figure 2.2-3 2-D View of typical maximum moment distribution in slab for k_s of 25 pci (Center Lift).

The influence of the variation of the Modulus of Subgrade Reaction, k_s, on the maximum differential deflection (maximum deflection minus minimum deflection) and the absolute value of the maximum moment follows:

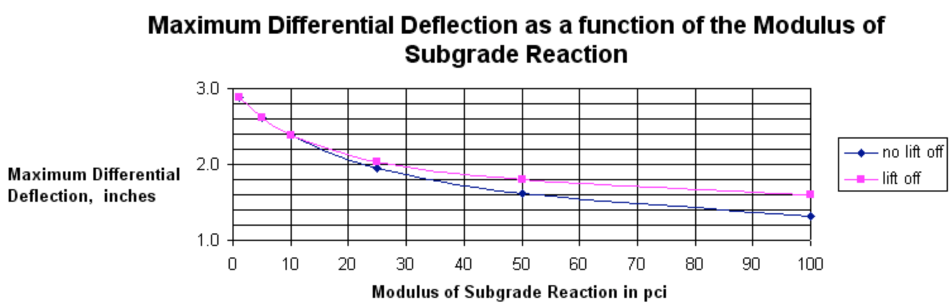

Figure 2.2-4 Maximum differential deflection as a function of Modulus of Subgrade Reaction in pci (Center Lift).

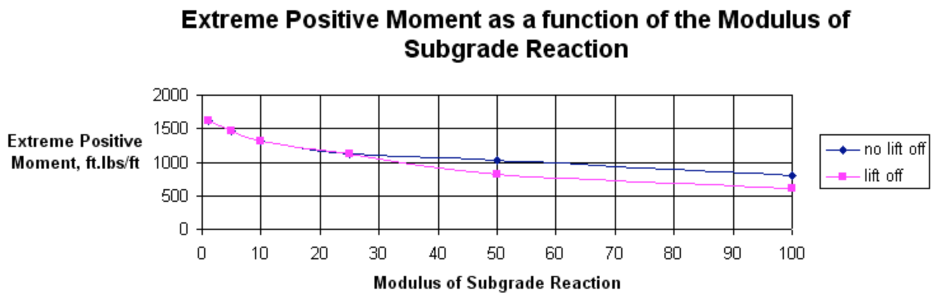

Figure 2.2-5 Extreme positive moment as a function of Modulus of Subgrade Reaction in pci (Center Lift).

The two curves in the preceding figures represent the results of two possible ways of modeling: tension-compression springs and compression springs only, the latter of which is the more accurate and includes lift-off in some of its solutions. The FPA has made no effort to determine at exactly what value of k_s liftoff is expected, other than that it starts occurring in the range of 10 to 25 pci.

In Appendix A.1 of PTI Design Procedure, 3rd Edition, page 59, one can read: "β = relative stiffness length, approximate distance from the edge of foundation to point of maximum moment $(1/12)\sqrt[4]{(E_cI/E_s)}$, ft". The FPA used E_c=1,600,000 psi in the calculations above. According to Das [7], E_s can be estimated using his formula 4.51: $E_s=(k_s)(B)(1-\mu^2)$ where B is the width of the foundation. The Poisson's coefficient μ for medium clay can be found in his table 3.9, and is in the range of 0.2 to 0.5. The FPA used a μ of 0.4 and k_s of 10 pci in the calculations. The width of the slab is 48'. From Das' formula 4.51: $E_s=(k_s)(48 \times 12)(1 - 0.4^2) = 4838$ psi. Substituting gives $\beta = (1/12)\sqrt[4]{(1{,}600{,}000*156{,}694/4838)} = 7.07$ ft.

Hence, there appears to be little agreement between the location of the maximum moment estimated at β per PTI Design Procedure and the actual locations of maximum moments. The FPA found maximum moments towards the center of the slab or near the corners of the soil support. The maximum moments do not occur approximately at a distance of e_m plus β from the edge of the slab in the center lift condition, which can be visibly verified in Figure 2.2-3 where the observed distance of the maximum positive moment is approximately 4.5' as compared to a β value of 7.07 ft. This difference is caused by the calculation procedure that analyzes the slab as a two-way rather than a one-way slab.

For a reasonable k_s range of 1 to 10 pci the maximum differential deflection varies by 20% while the maximum moment in the slab varies by 23%. By assuming a set value for k_s one can expect a variation of major design variables by ±10% for a typical range of soil conditions.

For soils with k_s greater than approximately 25 pci, lift-off in center lift can be expected along with a decrease in the maximum moment and an increase in the maximum differential deflection.

The curves in the preceding figures indicate that a foundation on stiff soil, i.e., with a high value of k_s, can be designed much lighter than those with a less stiff soil, (i.e., with a low value of k_s).

Therefore, the variable k_s is an important input parameter, as β depends on it. This variable is used in the PTI Design Procedure, 3rd Edition, as the basis for the calculation of the deflection ratio through the use of the variable C_Δ such as shown in the PTI Design Procedure, 3rd Edition Table 6.2.

The FPA recommends that the PTI Design Procedure:

- *Provide guidelines on how to derive the value for the Modulus of Subgrade Reaction (k_s) compatible with the PTI procedure for the derivation of e_m and y_m as well as β for a given soil profile for the size of the footprint of the foundation design.*

- *Include a re-statement of the meaning of β.*

- *Provide clarification for the maximum length the engineer needs to consider in his calculations.*

2.3 MINIMUM FIELD INVESTIGATION PROGRAM AND REPORTING

A minimum field investigation program often does not include information on soil compressibility, consolidation, or a history of tree cover for the lot. All three may be important for the determination of the design differential movement of the foundation.

2.3.1 Settlement

Settlement is downward movement of an underlying supporting soil stratum due to loading above in excess of the bearing capacity of the soil below. When the vertical loads from above are in excess of the bearing capacity of the soil strata directly below the foundation, the foundation and superstructure move downward. Encompassed in settlement are a) the immediate elastic compression and distortion of granular or clay soil particles, b) slope instability, and c) the long-term consolidation resulting from gradual expulsion of pore water from voids between saturated clay soil particles. Settlement may occur in all types of soils.

Compression in soil is the result of the change in volume or size caused by a force. PTI Design Procedure, 3rd Edition, Section 3.2.2, expresses the need for an expansive clay analysis as well as the application of compressibility equations to derive the estimated total settlement in the center of the foundation.

Consolidation is a type of compression often occurring in low permeability soils. It is a de-watering process over a certain amount of time, and is controlled by the rate at which water is squeezed out of the pores. While compression may occur almost immediately, consolidation may continue for months or years, depending upon the permeability of the soil. The presence of trees may increase the consolidation because the roots tend to de-water the soil and cause subsidence.

In some states other than Texas, collapsible soils exist. This represents a settlement type of loading, but the settlement takes place rapidly when the soils are wetted. This results in much larger differential movement over much shorter distances. In such locations, expansive soil is not of concern. The PTI Design Procedure, 3rd Edition does not address this issue, and it is implied that the procedure can be used to determine the geotechnical parameters for this case also. It should be made clear that the PTI Design Procedure, 3rd Edition, does not apply for collapsible soils.

Because non-elastic settlements will occur in locations with under-consolidated clays, soil compression should be analyzed in addition to expansive clays. Therefore the geotechnical report should indicate whether or not the site is under-consolidated and how much settlement should be expected. Such statements are not included in typical geotechnical reports.

The FPA recommends that the PTI Design Procedure include guidelines on how to compute soil compressibility and guidelines for the minimum geotechnical testing on which these calculations should be based.

2.3.2 Aerial Photographs and Tree Removal

Although geotechnical information from borehole interpretations may indicate the presence of root fibers or other organic matter, PTI Design Procedure, 3rd Edition, Section 3.3, titled "Minimum Field Investigation Program", tasks the geotechnical engineer in that “A representative series of aerial photographs are recommended to identify site conditions before subdivision grading is initiated.” The presence of trees, channels and man-made ditches prior to the development of a housing area may indicate that differential movements caused by moisture migration in expansive soils should be expected. In a development area trees are typically removed and the cavities left by the root removal process are filled in. Channels and man-made ditches may be filled in with soil that has different properties than the in situ soil, potentially leading to differential swell or settlement. Aerial or satellite photographs may assist in the geotechnical engineer’s judgment on the applicable geotechnical parameters, the subdivision development engineer's judgment on the development design, and the foundation design engineer’s judgment on the appropriate foundation system design.

The removal of trees in expansive soils has been known for many years to cause heave if a foundation is built upon the root zone before the soil has time to re-hydrate and swell to the elevation it was prior to shrinkage caused by long-term tree root desiccation. According to tests by Dr. Biddle [5], after tree removal in an expansive clay the soil can re-hydrate in as little as one year, though often longer, and the rate of soil heave drops significantly afterwards, depending on moisture availability. Movement may be noticeable for ten or more years.

Figure 2.3.2-1 shows a typical example of a stump removed from this site prior to construction.

Figure 2.3.2-1 Tree stump removed from beneath the building footprint.

In practice the removal of a large tree under the footprint of a house may lead to unexpected heave of several inches or more in the tree's former location under the slab. It is important for the geotechnical engineer to incorporate the effect of tree removal in the geotechnical design parameters. Aerial photographs are often the only available option to obtain tree information. Situations such as depicted in Figure 2.3.2-2 may be of particular concern because a mature tree may have been removed from under the footprint of the new house just prior to the construction activities. The lack of sufficient time between tree removal and construction may lead to distress of the residence, caused by unexpected localized swell of the soils under the foundation.

Figure 2.3.2-2 Presence of a logging truck prior to foundation construction.

Free aerial photographs suitable to determine if large trees were present on the location can be found on the internet and many web sites offer fee based aerial photographs recorded at different times and at higher resolutions.

The FPA recommends that the PTI Design Procedure specify:

- *That geotechnical testing criteria be provided to validate the moisture conditions of the lot to determine if it is ready for construction, and*

- *The additional measures that need to be taken in areas where trees have been removed prior to construction. Such recommendations may include a waiting period between tree removal and construction, or artificial rehydration of the soil strata.*

2.4 THE VALUE OF e_m AND y_m

Compared to the geotechnical reports based on PTI Design Procedure, 2nd Edition, current geotechnical reports provide e_m values that are often substantially larger for the same soil conditions. The probable explanation is that for values of α or α' (Unsaturated Diffusion Coefficient, where the primed value is modified for the soil fabric factor) in excess of 0.003 for center lift and 0.009 for edge lift, the e_m values increase to over 8' with a maximum value of 9'. The FPA has also observed a similar increase in y_m values.

The FPA observed that for the Houston area recent geotechnical reports typically give e_m edge lift values ranging from 4' to 9', with values in other areas of Texas for y_m of up to 5 inches for both center lift and edge lift conditions. While the typical values do not appear to vary numerically over a large range, the changes have a significant effect on the moments, shears and deflections used by the foundation engineer to design the foundation.

To illustrate the dilemma for designers, the FPA ran the VOLFLO Win computer program, Versions 1.0 (Build 052902) [22] and 1.5 (Build 041405) [23], to derive the values for e_m and y_m by using the input file for the soil profile of "Example Soil 1" that was provided with the VOLFLO Win 1.0 program. This soil profile represents soft and expansive soil. As the VOLFLO Win 1.5 demonstration software was distributed together with PTI Design Procedure, 3rd Edition, the FPA assumes there is a tacit acceptance and endorsement by the PTI of that software.

The FPA used "Calculate per Modified PTI Design Procedure" in the VOLFLO Win 1.0 program input for all layers to remove as much user influence on the results as possible. Similarly the FPA used the option "Determine per PTI 3rd Edition Manual Charts" in VOLFLO Win 1.5 program input and "Default Dry Design Envelope" and "Default Wet Design Envelope" as suction envelopes for edge lift and center lift calculations.

Table 2.4-1 illustrates the increase of e_m:

e_m and y_m for Example Soil 1					
Soil Support Condition	Software:	VOLFLO Win 1.0		VOLFLO Win 1.5	
	Case #:	1	2	3	4
		Per PTI 2nd Edition	Per Modified PTI Method	Per PTI 2nd Edition	Per PTI 3rd Edition
e_m center lift (ft)		5.9	9.0	5.9	9.0
e_m edge lift (ft)		3.1	5.3	3.1	5.3
y_m center lift (ft)		3.0	2.9	3.0	3.0
y_m edge lift (ft)		4.0	4.0	4.0	4.0

Table 2.4-1 e_m and y_m computed for "Example Soil 1" using different versions and options of GTK's VOLFLO Win program.

Notes:

1) VOLFLO Win 1 performs the calculation procedure as per PTI Design Procedure, 2nd Edition. The calculation results are shown in Case #1 of the Table 2.4-1.

2) The Modified PTI Design Procedure is a predecessor to the current VOLFLO Win 1.5 method with the updated geotechnical procedures anticipating PTI Design Procedure, 3rd Edition. It leads to more conservative e_m results than PTI Design Procedure, 2nd Edition, as can be seen in the calculation results shown in Case #2 of Table 2.4-1.

3) VOLFLO Win 1.5 performs the calculation procedure as per PTI Design Procedure, 2nd Edition as well as according to PTI Design Procedure, 3rd Edition. The results shown in Case #1 and #3 in Table 2.4-1 agree.

4) VOLFLO Win 1.5 results, based on PTI Design Procedure, 3rd Edition, are shown in Case #4 of the Table 2.4-1. These results clearly show that these e_m results are more conservative than those based on PTI Design Procedure, 2nd Edition.

There is no user manual to the GTK VOLFLO Win programs to describe how the formulae are interpreted in the program or how the input variables should be chosen to derive optimal results. The FPA is not aware of any independent publication on the applied theories verifying the curves and associated underlying formulae in PTI Design Procedure, 3rd Edition.

According to Section 5.3 of the "Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils" [15] (a PTI document distributed with PTI Design Procedure, 3rd Edition), e_m and y_m may be calculated according to PTI Design Procedure, 2nd Edition, as well as PTI Design Procedure, 3rd Edition. Thus all solutions quoted above are acceptable. Therefore, one may choose the most economical solution.

It should be noted that the PTI Design Procedure, 3rd Edition, has the following limitations:

- The edge moisture variation distance, e_m, is limited to 9 feet, and
- The maximum unrestrained differential soil movement, y_m, is limited to 4 inches.

If these parameters are exceeded, the design procedure in the PTI Design Procedure, 3rd Edition, are not valid and other methods such as FEM analysis should be used, or an alternative foundation system should be used.

For the above values of e_m and y_m, the impact to the foundation and the design procedure used is exemplified in the table below. The parameters from example "Appendix A.6.pti", a data file included with the PTSLAB 2.0 program, were used to determine what differences would develop using PTSLAB 2.0 [16] (generally used with PTI Design Procedure, 2nd Edition) and PTSLAB 3.0 [17] (generally used with PTI Design Procedure, 3rd Edition).

To simplify the comparison, the size of the slab was changed to 42'x42', the number of ribs was changed to 7 in each direction with a width of 12" and all ribs were identical. The e_m and

y_m values were changed as shown in Table 2.4-1, varying only for the rib depth until the design passed. Again, as PTSLAB 3.0 was distributed with PTI Design Procedure, 3^{rd} Edition, the FPA assumes there is an implied endorsement by the PTI of that software. Table 2.4-2 illustrates the results.

Beam Depth as a function of e_m and y_m						
		Depth (in.) of 12" Wide Beam Required to Pass Each Criteria				
		Software:	VOLFLO Win 1.0		VOLFLO Win 1.5	
		Case #:	1	2	3	4
Software	Criteria		Per PTI 2^{nd} Edition	Per Modified PTI Method	Per PTI 2^{nd} Edition	Per PTI 3^{rd} Edition
PTSLAB 2	Bending Stress		36	53	36	53
	Shear		34	39	34	39
PTSLAB 3	Bending Stress		35	52	35	52
	Shear		34	39	34	39
	Cracked Moment Capacity		100+	100+	100+	100+

Table 2.4-2 Rib depth required for a passing design solution as a function of y_m and e_m from Table 2.4-1.

Notes:

1) Based on the above rib depths, the calculation results show "all values within allowable limits".

2) For all solutions based on PTSLAB 3.0 the program reported "Cracked Moment Capacity in short direction: could not be calculated" and "Cracked Moment Capacity in long direction: could not be calculated". No reason is printed, no suggestion is offered.

3) The solutions demonstrate the effects on the beam depth required for different e_m and y_m soil parameters. Designers would not consider using beams in excess of 48"; they would use different beam and slab configurations for real solutions.

The majority of the post-tensioned slabs during the late 1990s and early 2000s were designed with PTSLAB2 and soils modeled as per PTI Design Procedure, 2^{nd} Edition. It follows from the examples above that a change to PTSLAB 3 and soil design parameters determined as per PTI Design Procedure, 3^{rd} Edition, will lead to heavier foundation designs. Because of the high success rate of foundations designed and constructed in accordance to PTI Design Procedure, 2^{nd} Edition, and the additional cost to build a foundation designed in accordance to PTI Design Procedure, 3^{rd} Edition, builders may continue to resist the proposed changes. Perhaps a cost-benefit analysis of the new vs. the older procedures could provide input for the new PTI Design Procedure.

The FPA recommends that the PTI Design Procedure require that geotechnical reports supply e_m and y_m values for both the PTI Design Procedure, 2nd Edition, and the PTI Design Procedure, 3rd Edition.

2.5 ALLOWABLE BEARING CAPACITY

According to PTI Design Procedure, 3rd Edition, Sections 3.2.2 2 and 4.5.2.3, the allowable bearing capacity may be applied to the bottom of the ribs and a portion of the slab. The allowable bearing capacity depends on the depth and the width of the strip over which the soil is loaded. This complicates the specification of the allowable bearing capacity and currently most geotechnical reports do not address the allowable bearing capacity at the bottom of the slab.

The FPA recommends that the PTI Design Procedure state that the geotechnical engineer specify the allowable bearing capacity as an average value (of the slab and ribs) that may be used for the total area of the ribs and a portion of the slab.

2.6 THE α FACTOR

The "unsaturated diffusion coefficient", α, is a measure of moisture movement in unsaturated soils. Its derivation and use can be found in PTI Design Procedure, 3rd Edition, in Appendix A.3.1, Section 9. In conventional geotechnical practice this variable is rarely measured and is usually assumed. Its value can be derived from psychrometer measurements. For a particular soil a vertical sample would be obtained. By inserting psychrometer probes at certain distances, a measure for the diffusion coefficient can be obtained.

Drs. Robert Lytton and Rifat Bulut, both of Texas A & M University, discussed the tools and the measurement procedures during their 20 Aug 03 presentation to the Foundation Performance Association [7]. The diffusion coefficient measured is for vertical water migration because of the nature of the test and the alignment of the soil sample.

In a follow-up discussion between the FPA and Dr. Lytton, it was suggested that the unsaturated diffusion coefficient in the horizontal direction may be one or two orders of magnitude larger than in the vertical direction. This is understandable as the clay materials are deposited in a layered fashion. In practice this means that the moisture can migrate in the horizontal direction much faster than what follows from the conventional psychrometer tests used for foundation design. The practical meaning is that e_m may be considerably underestimated when α is measured by the conventional procedure based on vertical soil samples.

The FPA recommends that the PTI Design Procedure include standard measurement procedures to measure α or issue a table of values as a function of soil properties as a guide so that the calculation of α is uniform within the geotechnical industry.

3.0 CONSIDERATIONS FOR STRUCTURAL CALCULATIONS

The FPA raised several concerns regarding calculation details listed in PTI Design Procedure, 3rd Edition. Recommendations were prepared to address some of these concerns, but for others the FPA concluded that more research is necessary.

3.1 LEVEL OF PRESTRESS

Section 2.2 of PTI Design Procedure, 3rd Edition, recommends a minimum prestress of 50 psi. A 100 psi prestress is indirectly referenced in the PTI 3rd Edition, Section 2.2 where it states “cracking can be mitigated by increasing the minimum prestress force to 0.10A”, typically assumed to be equivalent to 100 psi prestress times the area of gross concrete cross-section, A. Per “Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils” [15], Section R4.3.3, the units for 0.10A are kips. In summary, PTI Design Procedure, 3rd Edition, relates the choice of the prestress level to anticipated concrete shrinkage cracks.

The American Society of Civil Engineers’ “Recommended Practice for the Design of Residential Foundations” [3], Section 5.2.2.3(b), recommends a minimum residual average prestress of 100 psi. Apparently the experience of the design community has led to a more restrictive stand on the recommended prestress than the recommended prestress stated in PTI Design Procedure, 3rd Edition. From a practical point of view there must be an optimum value for the average prestress for a given design when the design is optimized for economy. For long slabs it may be better to specify minimum residual average prestress for a given location, such as 6β from the edges, in view of the friction losses that will be incurred.

The FPA recommends that the PTI Design Procedure follow ASCE Texas Section's minimum residual average prestress of 100 psi and clearly state such rather than specifying the prestress indirectly as a pretension force. As a refinement, the FPA recommends that the PTI Design Procedure consider specifying minimum residual average prestress for a given location, such as 6β from the edges, in view of the friction losses that will be incurred.

3.2 DESIGN EQUATION DISCONTINUITY

PTI Design Procedures were developed based on calculations made by Dr. W. Kent Wray for his Ph.D. dissertation in 1978 [24]. According to information available to the FPA these calculation procedures were based on finite element technology and procedures of that day and advances proposed by Dr. Wray. Statistical interpretations of similar solutions led to the parametric solutions still embraced by the PTI.

In practice this means that differences exist between the appropriate design solution and the PTI parametric solution. For an almost square slab there is a discontinuity in the solutions relating to the e_m calculation. PTI Design Procedure, 3rd Edition, Section 4.3.2, states, "There is a discontinuity in the equations for long direction center lift moments at e_m =5 ft (Eq. 6-14, 6.8.1.1). The moment for e_m, slightly greater than 5 ft is often less than the moment with e_m exactly equal to 5 ft. The curve fitting process used to arrive at the moment equations influences the discontinuity."

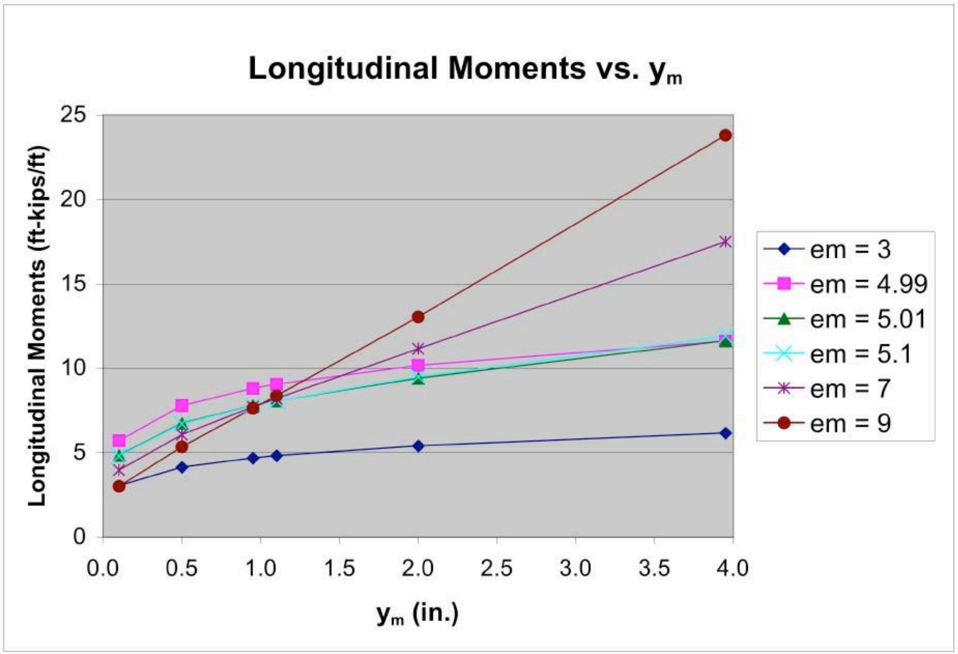

Figure 3.2-1 Longitudinal Moments vs. y_m
Comparisons for 40'x50' Slab (b=12", h=30", P=1000 plf, s=10', e_m is in feet)

As can be seen in Figure 3.2-1, for values of $y_m \leq 2.0$", the longitudinal moments per foot for e_m = 5.01' are 10% to 20% less than for e_m = 4.99', rather than approximately equal. For $y_m \geq$ 3.5", the moments per foot are slightly larger for e_m = 5.1' compared to e_m=4.99'. Logically, the moments per foot for e_m=5.01' should be less than the e_m=9', however, the plot shows that when y_m is $\leq$ 1.2", the moments per foot are lower, and they are significantly lower if $y_m \leq$ 0.5".

There is no reason that this discontinuity should exist today. The FPA is not aware of any studies that confirm the need for this discontinuity or its design consequences. There is no information available on the accuracy and continuity of the curve fitting for the parameter values on which the PTI method relies. The discontinuity quoted above indicates that the accuracy of the curve fitting may cause issues with the PTI solutions.

The FPA recommends that the PTI Design Procedure rewrite the equations to eliminate the discontinuity which occurs at e_m=5.

3.3 SHAPE FACTOR

In PTI Design Procedure, 3rd Edition, Section 4.5.1, the "Shape Factor" is used to determine if a slab can be accurately analyzed by a procedure relying on rectangles. According to PTI Design Procedure, 3rd Edition, Section 6.3, "Long narrow rectangles may not appropriately model the overall foundation and generally should not govern the design." A range where a certain shape and size becomes unacceptable may be more appropriate.

According to PTI Design Procedure, 3rd Edition, Sections 4.5.1 and 6.3, the Shape Factor, SF, defined below, should not exceed 24.

$$SF = \frac{Foundation\ Perimeter^2}{Foundation\ Area} \leq 24$$

As a simple example, for a 100’x50’ foundation, SF is 18, but for a 100’x25’ foundation, SF is 25. The user is advised that for SF greater than 24, either the foundation and/or construction plans should be revised, or finite element procedures should be used. The FPA has no access to data that would suggest a limiting value of 24.

The FPA recommends that the PTI Design Procedure justify how the limiting value of 24 for the Shape Factor criterion was determined. Alternatively, the FPA recommends that PTI define a different procedure for the engineer to use in order to determine if the PTI design procedure is applicable for a specific foundation plan.

The FPA recommends that the PTI Design Procedure should specify a range for the Shape Factor where engineering judgment must be used to decide if a PTI solution is applicable.

3.4 COMPRESSIBLE SOILS

Section 3.2.2 of PTI Design Procedure, 3rd Edition, includes a discussion on the compressibility of soils. Consideration 3 defines compressible soils. It concludes, “If the applied average pressure does not exceed the pre-consolidation pressure, for a depth within 0.85 the width of the entire foundation, it is unlikely that the site is compressible”. The PTI provides no guidance on what to do if consolidation is expected for expansive soils.

PTI Design Procedure, 2nd Edition, addresses compressible soils in an example in PTI Design Procedure, 2nd Edition, Appendix A.8. In this example the expected settlement is treated as an estimate for the maximum differential soil movement or swell, y_m. There does not seem to be a theoretical basis for this procedure.

In PTI Design Procedure, 3rd Edition, Section 6.13.3, Slab-on-Ground Constructed on Compressible Soils, the last sentence states, “…y_m shall be taken as the estimated differential settlement.” However, no method is presented for the determination of the estimated differential settlement.

The FPA recommends that:

- *If the PTI Committee considers that the effects of soil compression are significant, then clarification should be added to the PTI Design Procedure regarding how these effects should be included in the calculations for expansive soil.*

- *The PTI Design Procedure include calculation examples explaining the procedures for compressible soils, including how to determine estimated differential settlement, and how these procedures modify the standard calculations for expansive soils.*

3.5 RIB WIDTH

Some designers increase the width of the ribs to add bearing capacity. This is usually utilized when soft soil conditions exist or in the case of heavy wall loads. PTI Design Procedure, 3rd Edition, Section 4.5.2.3, states that within the rib width range of 8 to 14 inches "the flexural design is virtually unaffected by the rib width", i.e., the rib width does not have a major influence on the stiffness of the slab. From interpretations of PTI Design Procedure, 2nd Edition, Appendix A5, one could conclude the opposite.

As an example, the FPA computed the stiffness of a 4" thick ribbed slab 60' long with 5 equally sized ribs 32" deep (measured from the top of the slab). The FPA varied the width of all ribs from 8" to 48". The FPA considered the flange width at the ribs as per the ACI 318 [1] specifications. The consideration is made on this basis because the effective flange width of a rib is indirectly included in a finite element analysis on which the PTI solutions are based. The variation in the moment of inertia for the example slab is depicted below. In summary, adding width to the ribs does increase the stiffness of the slab, but not as effectively as increasing the rib depth.

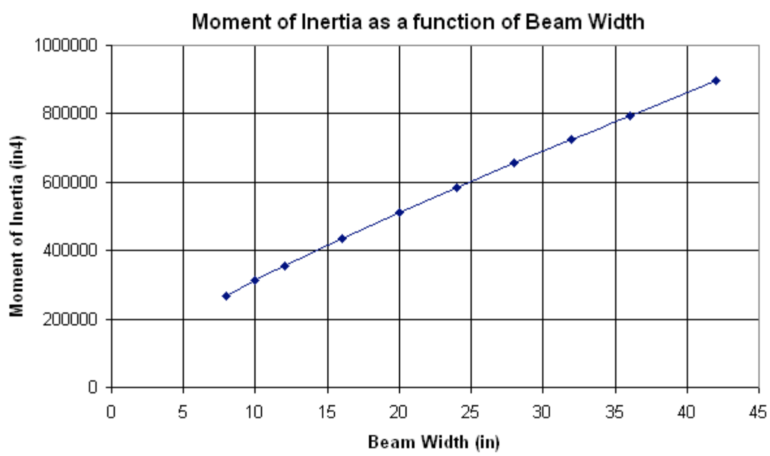

Figure 3.5-1 Variation of the moment of inertia with rib depth derived using the effective slab width as specified by ACI 318 for the foundation design described above.

It can be seen from the above plot that an increase in the foundation moment of inertia can be expected when the rib width is increased, contrary to what is stated in PTI Design Procedure, 3rd Edition, Section 4.5.2.3.

The FPA recommends that the PTI Design Procedure include the full section properties of ribs including those with a width in excess of 14 inches.

3.6 EFFECTIVE SHEAR TRANSFER AREA

PTI Design Procedure, 3rd Edition, Section 4.5.5 states that the "Applied shear stress is based upon the area of the ribs only, excluding the portion of the slab outside the width of the rib", which is conventional and conservative, because the flanges influence the effective moment of inertia and the location of the neutral axis.

For a typical rib connected to a slab the difference in shear transfer area between the ribs only and the rib plus the contributing slab width may be illustrated as in Figure 3.6-1.

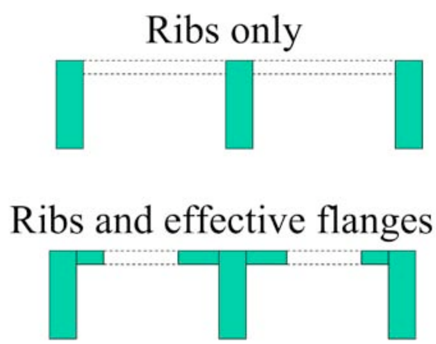

Figure 3.6-1 The vertical shear transfer area.

The FPA undertook a verification of the PTI allowable shear formula to determine the difference between the computed shear stress for a typical slab when flanges are included and when flanges are ignored. The FPA ignored the possibility that the shear in the vertical surface that connects the flanges to the ribs could be larger than the shear at the geometrical neutral axis. Although the derivation of a formula for the maximum shear is straightforward, we have not made an effort to present these calculations. These shears may govern in some cases.

As an example, the FPA considered a 40' wide slab with 5 ribs in the long direction. All internal ribs are 8" wide, and both edge ribs are 10" wide. All ribs are 28" deep, measured from the top of the slab. The slab is 4" thick. The effective flange width included as part of the rib can be computed with the rules provided by ACI 318-02, Sections 8.10.2 and 8.10.3. This leads to a total effective flange width of 284", including the ribs. For this example the FPA ignored post-tension.

The FPA used formula 8.1.2 listed in Roark's Formulas for Stress and Strain [25] to determine the shear stress τ at the neutral axis for the effective section caused by a vertical shear V at the section:

$$\tau = \frac{VA'y'}{Ib}$$

where: A' is the area of that part of the section above (or below) the horizontal plane where the shear stress is computed, y' is the distance from the neutral axis to the centroid of A', I is the moment of inertia of the section of the beam with respect to the neutral axis and b the width of the rectangular rib.

The FPA derived the shear for the slab described above for the case where both the ribs and the flanges are effective. This led to the following shear stress value based on the following geometric properties of the ribbed slab in the long direction:

- A' = 847 in^2
- Neutral axis is 8.74" below the surface of the slab
- The static moment of the portion above the neutral axis to the neutral axis is 8157 in^3
- y'=9.63"
- I = 159467 in^4
- b = 44"

This gives the following shear stress when the effective flange width is included as part of the rib:

$$\tau = 0.0011624 \text{ V.}$$

Similarly, had the FPA only used the ribs (no flanges) such as the PTI 2004 suggests, the FPA would have obtained the following geometric properties and shear stress value:

- A' = 616 in^2
- Neutral axis is 7.0" below the surface of the slab
- The static moment of the portion above the neutral axis to the neutral axis is 4312 in^3
- y'=7.0"
- I = 90490 in^4
- b = 44"

$$\tau = 0.0012175 \text{ V.}$$

The case where ribs only are considered leads to a 4.7% higher computed shear stress. Therefore, this difference indicates that using the ribs only for shear transfer is conservative. This conclusion is used in the following sections where the allowable stress is computed for a post-tensioned foundation utilizing ribs only.

The FPA agrees with the assumption by PTI that to use only the stiffening ribs in shear capacity calculations is conservative and no change is recommended.

3.7 ALLOWABLE SHEAR STRESS

The allowable concrete shear stress is quoted by PTI Design Procedure, 2nd Edition, Section 6.5 (D), Formula 10, and PTI Design Procedure, 3rd Edition, Section 6.5.4, Formula 6-7, as:

$$v_c = 1.7\sqrt{f_c^{'}} + 0.2 f_p$$

PTI defines v_c as the allowable concrete shear stress (psi) and f_p as the minimum average residual prestress compressive stress (psi). Note that f_p is positive although it is a compressive stress. The FPA has not followed this but used the conventional definition in the derivations where by compression is negative.

There is no reference or justification for use of this formula in PTI Design Procedure, 2nd Edition or 3rd Edition, in other codes, or in the literature, other than a statement that the "Committee researched the relationship between the vertical shear stress and the principal stress, documented recommended values…" in PTI Design Procedure, 3rd Edition Section

4.5.5. The FPA made an attempt to justify the use of this formula by attempting to derive it. With the help of Dr. Lytton of Texas A&M the FPA was able to re-derive the formula. In our derivation, the foundation is in edge lift, and as tension below the neutral axis of the rib is the controlling mode in edge lift, only the bottom of the rib is considered. The following assumptions were made:

1) There are no cracks in the concrete, so it can be considered as an elastic material.

2) All shear is carried by the ribs.

3) The specified tensile strength of concrete f_{ct} is calculated according to the ACI 318.-05 section 18.3.3 (Class "U", i.e., "Uncracked"): $f_{ct} = 6.0\sqrt{f_c^{'}}$ for two way slabs and $f_{ct} = 7.5\sqrt{f_c^{'}}$ for the stiffening ribs. PTI Design Procedure, 3rd Edition, Section 6.5.1, Formula 6-3, only allows $f_{ct} = 6.0\sqrt{f_c^{'}}$. Because shear capacity is limited to the stiffening ribs, it appears PTI's allowable is less than the industry accepted value for ribs. The FPA has generally followed the ACI nomenclature for the tensile strength f_{ct} for the specified tensile strength instead of PTI's f_t representing PTI's allowable tensile strength of concrete.

4) A safety factor of 1/0.45 is used in the derivation and relates to the allowable concrete flexural compressive stress listed in PTI Design Procedure, 3rd Edition, Section 6.5.1, Formula 6-4. There is no reference to such a value for a safety factor in ACI 318-02 or ASCE 7-98. The old alternate design methods (such as ASD) used an allowable extreme fiber stress in compression of 0.45 $f_c^{'}$, but the FPA could not find such factor for tension.

5) Mohr's circle is valid as a failure envelope for concrete.

The vertical stresses as well as the shear are zero at the bottom of the rib. This allows us to draw Mohr's circle for this condition as shown in Figure 3.7-1.

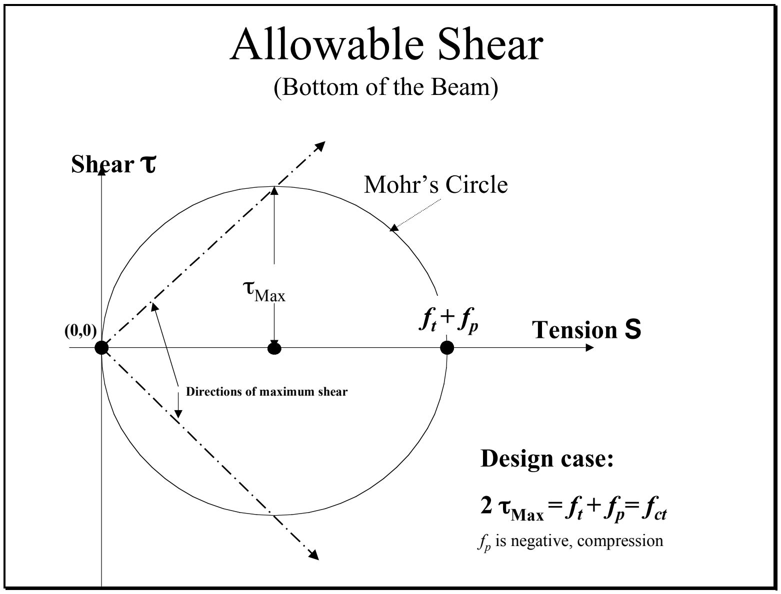

Figure 3.7-1 Mohr's circle at the bottom of the rib.

f_p = minimum average residual prestress compressive stress (from the post tension). f_p is negative (compression).
f_t = tensile stress caused by the bending moment at the rib surface.
τ_{Max} = maximum shear stress.

The shear at the bottom of the rib for the above condition must be smaller than τ_{Max}. From Mohr's circle it follows that τ_{Max} at the bottom of the rib is half the diameter of Mohr's circle. The diameter of the circle is $2\tau_{Max}$

Therefore,

$$2\tau_{Max} = f_t + f_p; \quad f_p \leq 0$$

$$\tau_{Max} = \frac{f_t + f_p}{2}$$

In the design case, τ_{Max} must be smaller than the allowable shear stress that can be related to the allowable tensile strength of concrete and the post-tension and a safety factor of 0.45. In the PTI formula derivation apparently the safety factor is included in f_p. Using PTI's built-in safety factor of 0.45, it follows:

$$\tau_{Max} = \frac{\left(f_t + f_p\right)_{Max}}{2}$$

$$\tau_{Allowable} = \frac{\tau_{Max}}{SF}; \quad \text{where } SF = \frac{1}{0.45} = 2.22$$

$$\tau_{Allowable} = \frac{7.5\sqrt{f_c^{'}}}{2}\frac{1}{SF} + \frac{f_p}{2 * SF}$$

$$\tau_{Allowable} = \frac{7.5\sqrt{f_c^{'}}}{2}\frac{1}{2.22} + \frac{f_p}{2 * 2.22}$$

$$\tau_{Allowable} = 1.7\sqrt{f_c^{'}} + 0.2 f_p; \; \text{ where } f_p \text{ is negative as per ACI convention}$$

Using the PTI nomenclature:

$$v_c = 1.7\sqrt{f_c^{'}} + 0.2 f_p \text{ (Note: } f_p \text{ is positive here, as defined by the PTI)}$$

Despite the identical formulae, v_c, the allowable concrete shear strength, is not related to $\tau_{Allowable}$ and therefore is not related to τ_{Max} because $\tau_{Allowable}$, as derived above, acts at ±45° and has no net vertical component.

We compared v_c, defined by PTI as “Allowable concrete shear stress, psi”, to what is allowed by other and older codes. In ACI-318-89, v_c is defined as “Permissible shear stress carried by concrete, psi”. In ACI-318-89, Appendix A, $v_c = 1.1\sqrt{f_c^{'}}$ for ribs and one way slabs, $1.2\sqrt{f_c^{'}}$ for joists, and up to $2\sqrt{f_c^{'}}$ for two way slabs depending on the aspect ratio. In ACI 318-02 Formula 11.7 one can find that nominal shear strength provided by the concrete can amount to over $3.5\sqrt{f_c^{'}}$. There seems little difference between how PTI and ACI defines v_c.

Assuming there is no other derivation of the PTI shear formula available, the FPA concludes the following:

- τ_{Max} has no relation to v_c, because as per Mohr’s circle there is no vertical shear component at or near the surface of the bottom of the rib. Because the derivation only concerns the surface and does not relate to the rest of the rib, the formula cannot be used for the derivation of the allowable shear capacity of the ribs.

- The safety factor of 0.45 relates to the “extreme fiber stress in compression” as defined in ACI 318-99, Section A3.1(a). This safety factor on which the above derivation is based has nothing to do with the “extreme fiber stress in compression”.

- It would be helpful for PTI to use the same nomenclature as ACI.

Generally, the maximum allowable shear stress is a function of the allowable tension. As stated above, the maximum allowable shear stress, $\tau_{Allowable}$, is proportional to $\left(f_t + f_p\right)_{Max}$ as:

$$\tau_{Max} = \frac{\left(f_t + f_p\right)_{Max}}{2}$$
$$2\tau_{Max} = \left(f_t + f_p\right)_{Max} = 7.5\sqrt{f_c^{'}}$$
$$2\tau_{Allowable} = \frac{2\tau_{Max}}{SF} = 7.5\sqrt{f_c^{'}}\frac{1}{SF}$$
$$\tau_{Allowable} = \frac{7.5\sqrt{f_c^{'}}}{2}\frac{1}{SF}$$

Because f_p is negative, f_t may increase as long as the algebraic sum $\left(f_t + f_p\right)$ stays below $\tau_{Allowable}$. Post-tensioning reduces the magnitude of the tensile stress in concrete that is caused by bending. The definition of f_t, the "Allowable concrete flexural tensile stress" in PTI Design Procedure, 3rd Edition, Section 6.5.1, Formula 6.3 is $f_{t_{Allowable}} = 6\sqrt{f_c^{'}}$. f_t could be interpreted to mean just the stresses caused by flexure, rather than the stresses caused by flexure plus prestress. The FPA does not believe this is the intended meaning because in PTI examples, such as PTI Design Procedure, 3rd Edition, Section A.3.2.3, it clearly means "Allowable Concrete Tensile Stress from Flexure, Post-Tension and Cable Eccentricity".

The FPA recommends that PTI Design Procedure adopt ACI's nomenclature for f_t as well as the definition and magnitude of the allowable concrete tensile stress.

3.7.1 Safety factor for Shear Calculations

ACI uses Load and Resistance Factored Design (LRFD) and PTI uses Allowable Stress Design (ASD) methods. How reasonable is this value when design load and material factors are applied according to the ACI 318 [1] or ASCE 7 [2] design loading conditions? For a one and a two story 60 ft. by 40 ft. house, the FPA used the following typical un-factored loads from an actual design in order to derive typical ASD safety factors:

	One Story Building Load Distribution (psf)	Two Story Building Load Distribution (psf)
Roof dead load	20	20
Ceiling dead load (under roof)	7	7
2nd story floor dead load	-	20
Foundation and walls	140	181
D=Total Dead Load	**167**	**228**
Attic live load	5	5
1st story live load	40	40
2nd story live load	-	40
L=Total Live Load	**45**	**95**
Roof live load	5	5
S=Total Roof Live Load	**5**	**5**
Total Load	**217**	**328**

Table 3.7.1-1 Load values for typical slab design

Load factors for typical slab design loading combinations to obtain the design loads (U) are specified in ACI 318-02 section 9.2.1. In particular, Formulas 9-2 and 9-3 apply to residential foundation loads. The FPA used the above load values and computed the equivalent ASD safety factors as follows:

LFRD Ultimate Load = U = Σ (Load Factor)(Actual Load)

$$\text{Equivalent ASD Safety Factor} = \text{SF} = \frac{\text{U}}{(\phi)(\text{Total Load})}$$

Where, according to the ACI 318-02, Section 9.3.2.3, the strength reduction factor for shear and torsion is $\phi = 0.75$, and the Total Load is as computed above.

	U	SF
ACI 318-02 Formula, One-Story House		
(9-2) : U = 1.2*D + 1.6*L + 0.5*S =	275	1.69
(9-3) : U = 1.2*D + 1.0*L + 1.6*S =	253	1.55
ACI 318-02 Formula, Two-Story House		
(9-2) : U = 1.2*D + 1.6*L + 0.5*S =	428	**1.75** (used below)
(9-3) : U = 1.2*D + 1.0*L + 1.6*S =	377	1.53

It appears that for a typical two-story house the weighted and combined load and strength reduction factor (material factor) could be as high as 1.75, lower than the value of 1/0.45=2.22 that PTI apparently applied. From the above derivation it is obvious that the safety factor will vary with the type and size of the building. For the purpose of the following derivations the FPA has assumed a value for the SF of 1.75.

3.7.2 Shear at the geometrical Neutral axis

The FPA has assumed that all shear is carried by the rectangular ribs, as derived in Section 3.7.1 of this document. For that case, the shear at the geometrical neutral axis is easily calculated from standard formulas such as printed in Roark's Formulas for Stress and Strain [25] Formulas 8.1-2 and 8.1-13 as:

$$\tau_{Max} = 1.5 v_c \quad \text{where: } v_c = \frac{V_c}{bh}$$

When no post-tension is present, Mohr's circle is a circle with radius τ about the origin of the coordinate system. The stresses on a vertical plane at the neutral axis is characterized by (0,τ). When post-tension (horizontal compression) is added, the radius of the circle increases and the center of the circle moves towards the compression side over a distance $f_p/2$ because the stresses on a vertical plane at the neutral axis become (f_p,τ) as illustrated in the following figure:

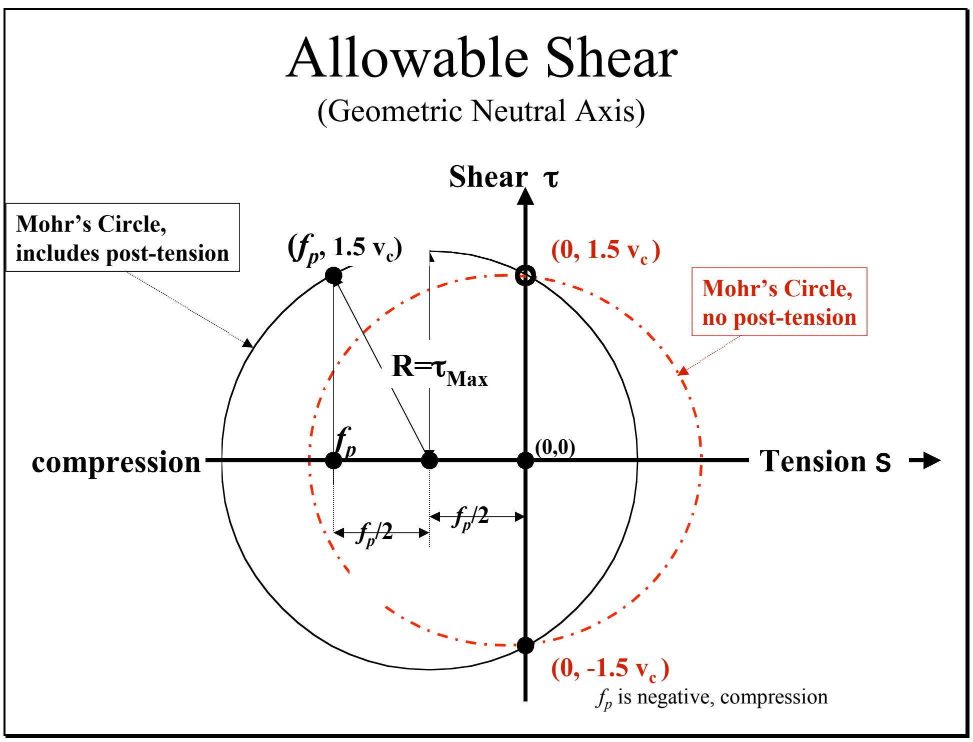

Figure 3.7.2-1 Mohr's circle for shear and post-tension at the geometrical neutral axis

The maximum shear for this case is the radius of the circle defined by the points (f_p, 1.5 v_c) and (0, -1.5 v_c) with a center at (f_p/2,0). From the Pythagorean theorem follows that:

$$\tau_{Max} = \sqrt{\left(\frac{f_p}{2}\right)^2 + (1.5 v_c)^2}$$

$$\tau_{Max} = R = \sqrt{\left(f_p\right)^2 + \left(1.5v_c\right)^2}$$

The maximum tension in the concrete must be positive but smaller than the allowable tension because otherwise Mohr's circle would be completely on the left of the ordinate axis and the above derivation of the maximum shear would not be valid. From Mohr's circle follows:

$$0 < \tau_{Max} + \frac{f_p}{2} < \frac{7.5\sqrt{f_c^{'}}}{SF}, SF = 1.75$$

The left side of the equation states that there must be some tension in Mohr's circle. This leads to the specification of a maximum value for f_p (note that f_p is negative):

$$0 < \tau_{Max} + \frac{f_p}{2}$$

$$0 < 1.5v_c + \frac{f_p}{2}$$

$$0 < v_c + \frac{f_p}{3}$$

$$f_p > 3v_c$$

Considering the right hand side of that equation:

$$\tau_{Max} + \frac{f_p}{2} < 4.3\sqrt{f_c^{'}}$$

$$\tau_{Max} < 4.3\sqrt{f_c^{'}} - \frac{f_p}{2}$$

From the Pythagorean theorem, the FPA found earlier:

$$\tau_{Max} = \sqrt{\left(\frac{f_p}{2}\right)^2 + \left(1.5v_c\right)^2}$$

Substituting for τ_{max} :

$$\sqrt{\left(\frac{f_p}{2}\right)^2 + (1.5v_c)^2} < 4.3\sqrt{f_c^{'}} - \frac{f_p}{2}$$

$$\left(\frac{f_p}{2}\right)^2 + (1.5v_c)^2 < 18.4 f_c^{'} + \left(\frac{f_p}{2}\right)^2 - 4.3 f_p \sqrt{f_c^{'}}$$

$$2.25 v_c^{\,2} < 18.4 f_c^{'} - 4.3 f_p \sqrt{f_c^{'}}$$

$$v_c < \frac{\sqrt{18.4 f_c^{'} - 4.3 f_p \sqrt{f_c^{'}}}}{1.5}$$

$$or:$$

$$v_c < \frac{\sqrt{(7.5/SF)^2 f_c^{'} - 7.5/SF * f_p \sqrt{f_c^{'}}}}{1.5}$$

This last formula is valid for $v_c + f_p > 0$ where $f_p < 0$**,** (f_p has a negative value**,** compression).

The presence of post-tension to typical levels increases the allowable shear at the geometrical neutral axis. This allowable shear force the ribs (slab) can carry is:

$$V_c = v_c bh$$

where *b* is the width of the rib(s) and
h is their depth measured from the top of the slab to the bottom of the rib(s).

3.7.3 Comparison of allowable shears

The FPA compared the allowable shear values for v_c and V_c using the formula derived above and the PTI formulation. Assuming $f_c^{'}$ = 2500 psi and f_p = -100 psi, (In PTI nomenclature: f_p = +100):

a) The bottom of the rib (PTI formula):
$v_{Max} = 1.7\sqrt{f_c^{'}} + 0.2 f_p = 105$ psi (has no meaning)

b) For the geometrical neutral axis of the slab using a safety factor of 1.75:

$$v_c < \frac{\sqrt{18.4 f_c^{'} - 4.3 f_p \sqrt{f_c^{'}}}}{1.5}$$

$$v_c < 173 \text{ psi}$$

Higher allowable shear values are found for shears computed at the neutral axis than is presently allowed by the PTI.

Calculations performed by the FPA to obtain shear values in the horizontal plane just below the slab indicate that the allowable shear is higher for those areas and thus less conservative using the following formula that can be derived similarly to the shear formula for the neutral axis:

$$v_c = \frac{2}{3}\sqrt{\left(\frac{7.5\sqrt{f_c^{'}}}{SF}\frac{t}{c} - f_p\right)\frac{7.5\sqrt{f_c^{'}}}{SF}}$$

where c is the distance from the top of the slab to the geometric neutral axis of the rib and t is the thickness of the slab.

The explanation for the higher allowable shear value is that post-tension reduces the shear in this area because of the large flange areas where the post-tension disproportionately reduces the shear that has to be transferred through the rib. The FPA did not investigate the allowable shear values in the vertical planes where the flanges connect to the rib.

The FPA concludes that the unreferenced and undocumented PTI formula for the allowable shear has room for improvement. A formula for the allowable shear is offered for consideration.

The FPA recommends that PTI accept the ACI industry accepted nomenclature and increase the allowable tension as listed in PTI Design Procedure, 3rd Edition, Section 6.5.1, Formula 6.3, to an industry accepted value of $f_{ct} = 7.5\sqrt{f_c^{'}}$.

3.8 DISCONTINUOUS RIBS

In PTI Design Procedure, 3rd Edition, Section 4.5.2.4, Rib Continuity, it is stated that ribs should be continuous between the edges of the foundation in both directions. If ribs are not continuous, certain requirements should be met. Specifically, "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." PTI does not provide any boundary conditions or guidelines pertaining to requirements (b) and (c). Requirement (c) is the most difficult requirement to analyze because the PTI design equations do not provide any guidelines for these computations. The FPA investigated how this requirement impacts foundation design.

Six simple ribbed slab layouts were defined by the FPA. The first example consists of a 37'x37'x4" slab with four continuous 12"x28" ribs in each direction spaced 12' o.c. (see Figure 3.8-1). The rib depths are measured from the top of the slab, per PTI convention. Concrete strength is specified as f'_c = 3000 psi, and its density is 150 pcf. The modulus of soil reaction was given a value of k_s=75 pci, typical for Houston. The perimeter ribs of the foundation were given a line load of 1000 plf to represent exterior wall loads; a conventional uniform (live) load on the slab of 40 psf was applied. The FPA used an edge moisture (e_m) variation distance of 5.5'.

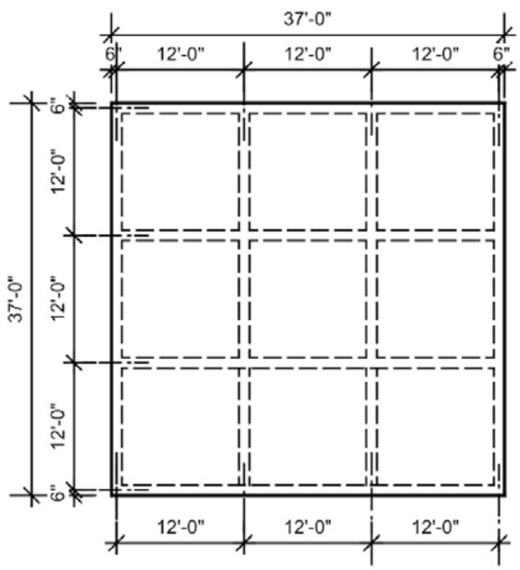

Figure 3.8-1 Problem 1 with 0' Rib Offsets.

Discontinuous ribs were simulated by offsetting the "vertical" ribs as depicted in the following figures.

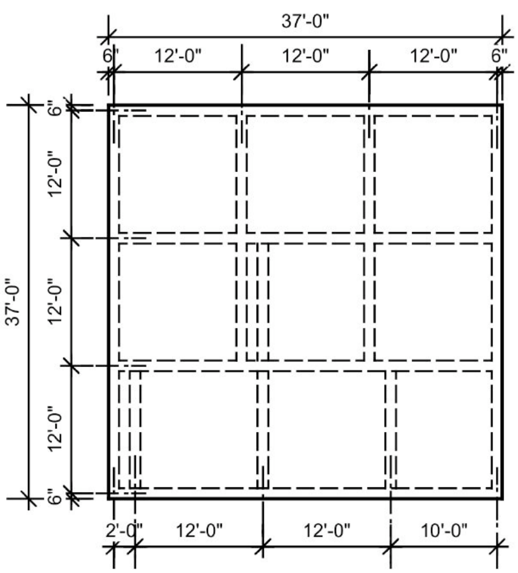

Figure 3.8-2 Problem 2 with 2' Rib Offsets.

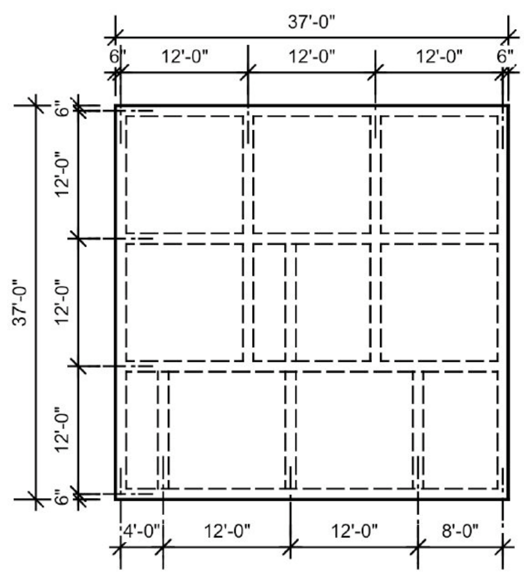

Figure 3.8-3 Problem 3 with 4' Rib Offsets.

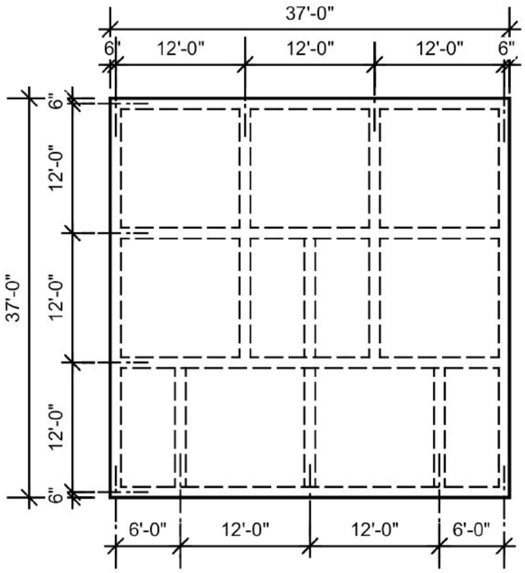

Figure 3.8-4 Problem 4 with 6' Rib Offsets.

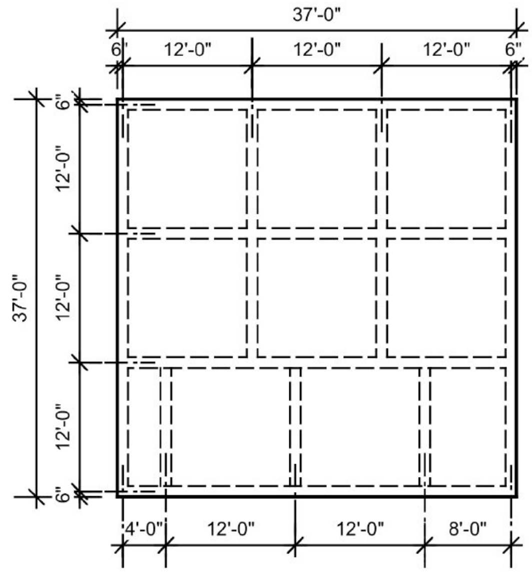

Figure 3.8-5 Problem 5 with 4' Rib Offsets and one rib shorter than in Problem 3.

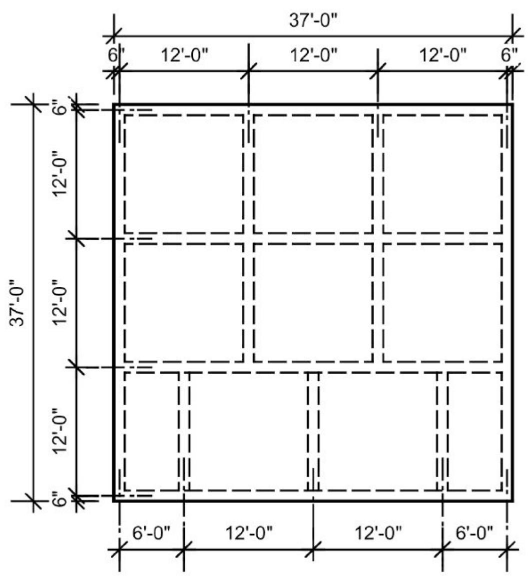

Figure 3.8-6 Problem 6 with 6' Rib Offsets and one rib shorter than in Problem 4.

For Problems 2 through 6, a rib was added in the lower left corner to limit the total span between the ribs. For Problems 2 through 4, a rib was added in the center to introduce a parallel rib as per PTI suggestions. The ribs were progressively moved to the right as depicted in Problems 3 and 4. The parallel center rib was removed from Problems 3 and 4 to form Problems 5 and 6.

"SAFE" Finite Element program, Version 8.0.6, was utilized for the finite element analysis of the above examples. This program does not model non-linear springs, it cannot accommodate horizontal loads such as those loads from post-tensioning, and it cannot accommodate soil support that start to act after the foundation has deflected a certain distance. The program is, however, able to simulate loading changes caused by lift-off from the soil.

Figure 3.8-7 Example of a discontinuous rib under construction.

The influence of post-tensioning forces was not included in the analysis. Because the ribbed slab is expected to respond linearly, the influence of post-tensioning forces could be added separately through superposition (not considered here). Post-tensioning is expected to exacerbate the negative effects of the discontinuity of the ribs because the discontinuous ribs cannot have post-tension reinforcing. A sketch of the soil support conditions considered is shown in Figure 3.8-8.

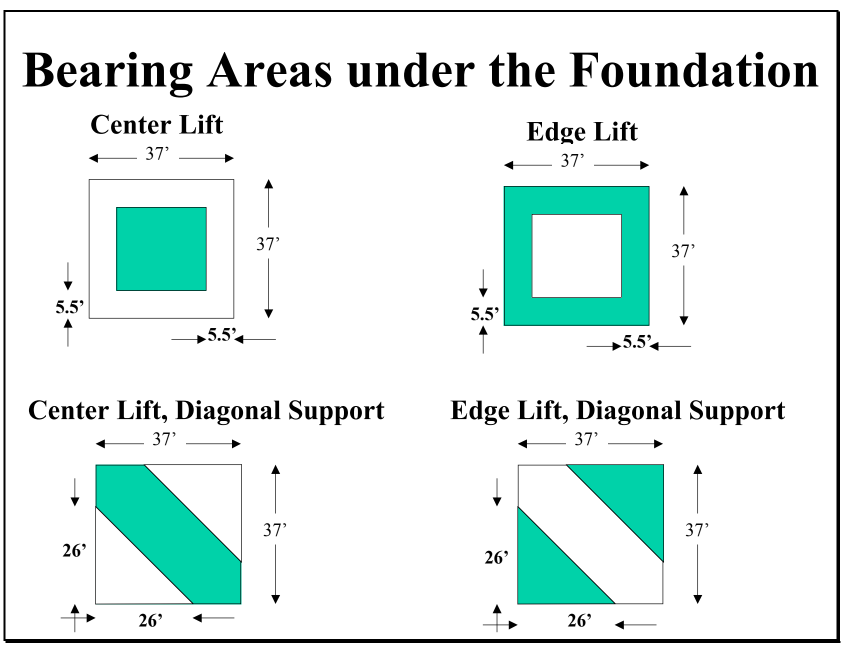

Figure 3.8-8 Soil Support Conditions (Shaded portion indicates soil support)

The calculation results for the six examples depicted in Figures 3.8-1 to 3.8-6 are summarized in Table 3.8-1.

Model	Slab M Max. (k-in/in)	Slab M Min. (k-in/in)	Slab M_{xy} Max. (k-in/in)	Slab |V| Max. (k/in)	Differential Deflection Max (in)	Ribs M Max. (k-in)	Ribs M Min. (k-in)	Ribs |V| Max. (k)	Ribs Torsion Max. (k-in)
EDGE LIFT									
Problem 1	1.61	-0.98	0.37	0.29	0.15	457	-306	7.60	149
Problem 2	**1.73**	-0.95	0.38	0.29	0.15	**591**	**-433**	**10.38**	**166.8**
Problem 3	**1.79**	-0.96	0.38	0.30	0.14	**666**	**-380**	**10.74**	**166**
Problem 4	**1.72**	-1.00	0.38	0.30	0.14	574	-325	**9.81**	153
Problem 5	**1.87**	-0.96	0.37	0.30	0.16	**594**	**-400**	**10.58**	**188**
Problem 6	**1.71**	-0.98	0.36	0.30	**0.18**	**467**	**-358**	**9.76**	147
CENTER LIFT									
Problem 1	4.81	-3.46	1.01	0.47	0.84	47	-1654	22.78	225
Problem 2	**5.22**	**-3.23**	**1.23**	0.42	**0.73**	**50**	**-2031**	**26.23**	**663**
Problem 3	**6.38**	**-2.93**	**1.21**	0.43	**0.69**	**55**	**-2112**	**27.96**	**394**
Problem 4	4.74	-3.39	0.98	0.47	0.84	**73**	**-1810**	**24.68**	**275**
Problem 5	**6.79**	**-5.23**	**1.25**	0.47	**0.72**	**101**	**-2064**	**28.44**	**566**
Problem 6	4.81	**-5.49**	0.99	0.49	0.82	**84**	**-1770**	**24.63**	**359**

Table 3.8-1 Calculation results for Problems 1 to 6.
Note: Bold numbers indicate a deviation of greater than 5% from the base case, Problem 1.

The PTI Design Procedure, 3rd Edition, does not set limits on how much difference in design values may be acceptable when the case with continuous and discontinuous ribs are compared. Thus the FPA assumed that a difference in design values of up to 5% may be acceptable. Table 3.8-1 summarizes calculation results for the selected range of support conditions and cases for discontinuous and continuous ribs; many of the computed differences for the selected design variables are outside of that range (bolded). In particular the differences computed for moments, shears and torsions for the ribs are reason for concern.

In the “BRAB” manual [21] center support as well as diagonal support must be considered. Such a support condition, in practice, does develop because of soil swelling effects caused by tree removal or plumbing leaks near an outside corner. We made calculations that simulate the equivalent support conditions for diagonal support for both edge and center lift support. BRAB suggests that these support conditions may well lead to more severe deflections. The same calculated design values from Table 3.8-1 were used for the edge lift and center lift cases. Theses results are compared to both diagonal support cases as defined in Table 3.8-2.

Calculation Results for Case 1 in Table 3.8-1 and Support Conditions in Figure 3.8-8					
Support Condition:		Edge Lift (Table 3.8-1)	Edge Lift Corner Support	Center Lift (Table 3.8-1)	Center Lift Diagonal Support
Slab	M max (k-in/in)	1.61	3.23	4.81	3.24
	M min (k-in/in)	-0.98	-1.92	-3.46	-3.46
	M_{xy} max (k-in/in)	0.37	0.96	1.01	1.25
	V max (k/in)	0.29	1.45	0.47	1.94
	Max Differential Deflection (in)	0.15	0.85	0.84	2.54
Ribs	M max (k-in)	457	265	47	185
	M min (k-in)	-306	-1560	-1654	-2975
	V max (k)	7.60	17.85	22.78	32.3
	Max Torsion (k-in)	149	333	225	643

Table 3.8-2 Calculation results for edge diagonal lift and center diagonal lift.

From these results it follows that the design values for deflection, torsion, and shear would control the slab design while the minimum moment, shear, and torsion computed for center diagonal lift control the rib design.

The results from the above examples agree with PTI's recommendations discouraging the use of discontinuous ribs. Additionally, the results demonstrate that the center or edge diagonal support cases could govern the design values for the foundation. These loading conditions are ignored in the PTI parametric design procedures.

The FPA recommends that PTI Design Procedure, 3rd Edition, Section 4.5.2.4, Rib Continuity (c) be deleted and replaced by the following sentence: "The use of design conditions where a parallel rib is connected to a parallel rib by a perpendicular rib which transfers the bending moment in the rib by torsion must be supported with calculations."

The FPA also recommends, based on the diagonal support cases above, that consideration be given to foundation support conditions for certain possible conditions such as sewer leaks, foundation lift, poor drainage, heave from felled trees, subsidence effects from maturing trees, etc.

4.0 CONSTRUCTION TOLERANCES AND LEVELS

In PTI Design Procedure, 3rd Edition, Section 8.3, it is stated that ACI 117-90 for construction levelness tolerance applies to slab-on-ground foundations. This is further discussed in the PTI Technical Notes Issue 9 of July 2000 [6] where one can find that the surface elevation of a slab must be within an envelope of 1-1/2" (that is, the actual elevation may be ¾" higher or lower than the specified elevation) and all elevation variations must fit within that 1-1/2" envelope. However, PTI does not specify when these measurements should be made. The following illustrates why the timing of this measurement is important.

In the collective experience of the FPA, many foundations are not within this tolerance level. However, in Texas, legislation by the Texas Residential Construction Committee became law after June 1, 2005. Page 4 (§304.2 (a) (2)) of their provisions [20] states that if no elevations of the foundation were taken prior to substantial completion of the residential construction project then the foundation for the habitable areas of the home are presumed to be level to ±¾" over the length of the foundation. The provisions also state (Page 52, §304.100 (a) (1)(A)) that tilt and deflection of a slab are to be compared to the original level measurements of the foundation.

In other words, if a homeowner wants to prove foundation distress, the homeowner compares new deflection or tilt measurements to the original level measurements, or, if not available, to an assumed levelness condition as per the TRCC rules. The TRCC does not suggest the timing of the level measurements for newly constructed foundations.

It should be noted that because flooring contractors typically level uneven slab surfaces prior to placing tile, wood and sometimes carpet, the level of the finished floors may not be a true representation of the levelness of the slab.

The FPA used the example slab in Appendix A.6 of PTI Design Procedure, 2nd Edition, to verify what deflections and deformations one may expect between the placing of the ribbed slab and the completion of the brick veneered residence.

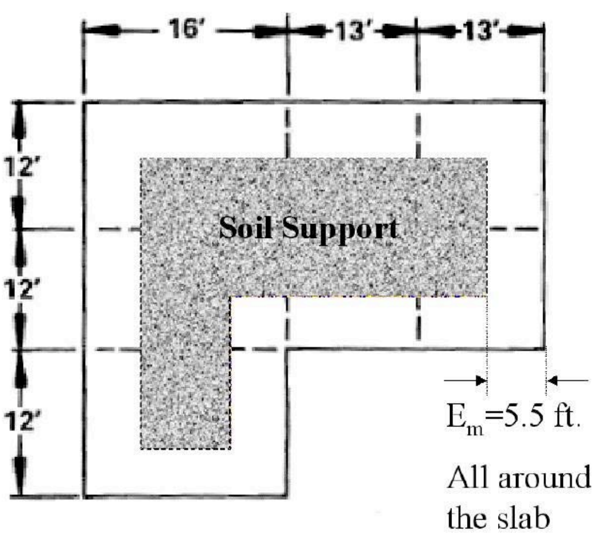

Figure 4.0-1 PTI foundation model from Appendix A.6 of PTI Design Procedure, 2nd Edition.

The FPA created a Finite Element model of the example slab and completed a Finite Element Analysis utilizing the SAFE program, Version 8. No post-tensioning was applied, because of program limitations and because post-tensioning creates additional deflections throughout the building process. These deflections depend on the eccentricity of the post-tension force, which can be either positive or negative depending on the design. The FPA used a deflection multiplier of 2 (ACI 318-02 9.5.2.5) to account for creep and curling of the freshly placed concrete. For various values of the soil subgrade modulus, k_s, see PTI Design Procedure, 3rd Edition Table 6.1, and ARMY TM 5-118-1 [4].

For the following example, a k_s value of 10 pci, for expansive clays, was used in the analysis. For the influence of the value of k_s on the deformations of a simple model foundation, see Section 2.2 of this paper.

Modeling limitations in the analysis presented include:

- Deflection multipliers for creep were not considered in the results. By not considering the deflection multiplier, the deflections in the following example will be underestimated.

- The support areas are limited to those areas defined as soil support. The following analyses are not exhaustive because no iterative calculations were made to include fitting the foundation to the dried soil profile. Because in actuality there may be additional soil support after the foundation deflects due to center lift, the deflections in the following example may be overestimated.

The following sequence of events is envisaged:

1) The slab is placed and the concrete cures. After post-tensioning, the slab is perfectly level. The slab is uniformly supported by the soil, and the soil will deform under the weight of the concrete but the deformation will be uniform and limited. These loads were estimated as 75 psf for the pad and 50 psf for the 4" concrete giving a total of 125 psf additional loading on the soil compared to its original loading condition without the foundation. This deformation is computed as 0.08 inches. The computed deformation is illustrated in Figure 4.0-2.

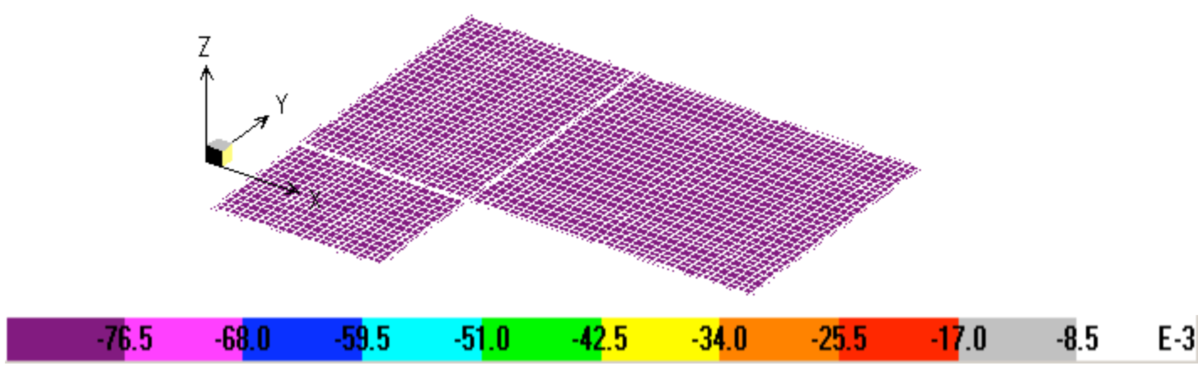

Figure 4.0-2 Settlement of slab after placement due to self-weight of concrete and pad above grade. The legend scale is in inches x 10^{-3}.

The computed deformation is assumed to be independent of the size of the slab because it is only dependent on the unit weight of the concrete and pad above grade and the modulus of subgrade reaction of the soil. Because of equipment limitations of typical level measurement devices, the surface deformations would probably not be measurable when compared to an external benchmark. We have ignored this settlement and the loading that caused it in our further calculations.

2) The brick veneer, walls, framing, finishes and roof (dead loads) are installed, causing an edge line load of 1.04 kips/ft as per the PTI example. This causes a maximum differential deflection of 0.33 inches. Actual measurements may show these deformations, although not accurately because of the tolerance of the measurement device. Under this loading condition, one could expect a contour plot of the deformed slab to look similar to Figure 4.0-3.

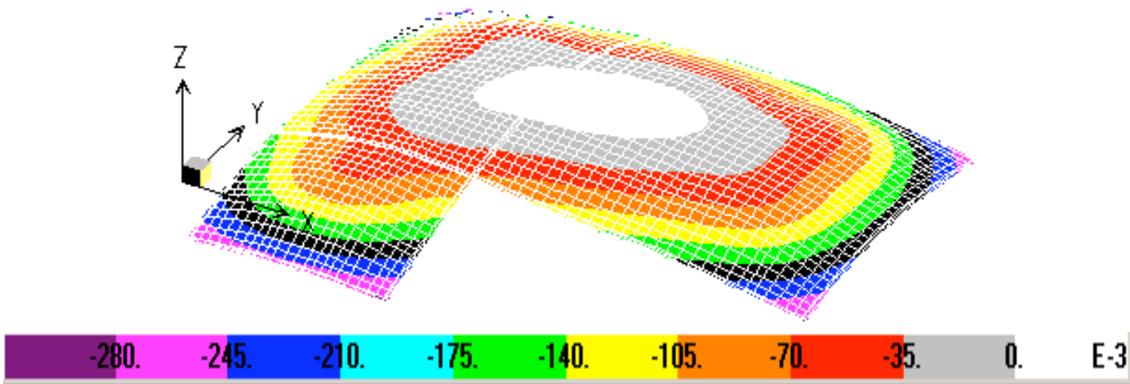

Figure 4.0-3 Model of foundation slab after edge dead loads are applied. Soil support is uniform under the complete slab. The legend scale shows inches x 10^{-3}.

The differential deflections (settlements) from installing the walls only will not exceed construction tolerances or the TRCC rules.

3) As the soil dries, the center lift condition governs (e_m=5.5') and the weight of the floor slab and ribs (average of 110 psf) and the edge loads along the edge of the slab is no longer carried by the soil. The maximum differential deflection increases from 0.33 to 3.08". Under this loading condition, one could expect a contour plot of the deformed slab to look similar to Figure 4.0-4.

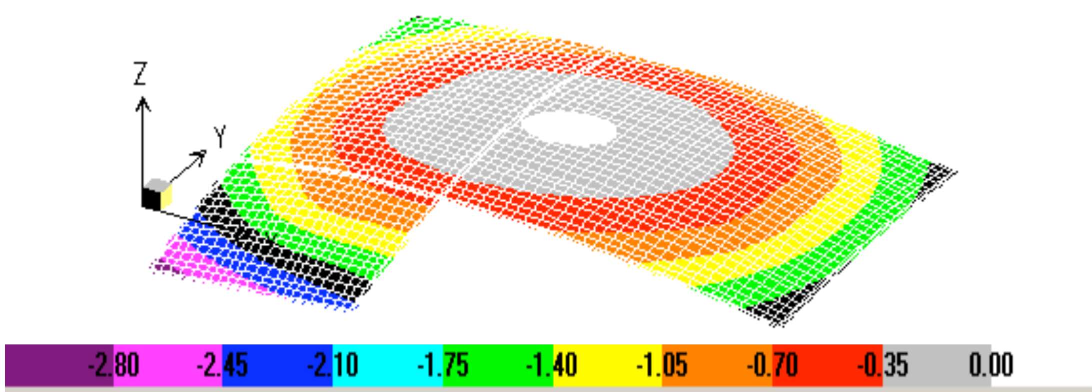

Figure 4.0-4 Model of foundation slab under center lift.
The legend scale is in inches.

As soon as center lift occurs the differential deflections due to the load of the superstructure and the cantilevered slab weight will cause ACI construction tolerances to be exceeded. If the deflection is greater than y_m, the deflected slab will keep in contact with the soil, thereby decreasing the actual deflections.

4) A tree is found at the re-entrant corner of the foundation as depicted below. It is assumed that the foundation was placed during a time when the foundation soils were wet and the tree did not draw a significant amount of water. It gets dry in the summer and the tree draws a significant amount of water out of the soil, causing the foundation to lose a certain amount of support in addition to what the edge moisture variation distance of 5.5' describes. e_m values are adjusted to reflect an approximate tree canopy area as shown in the following illustration. We have assumed a soil support area as follows:

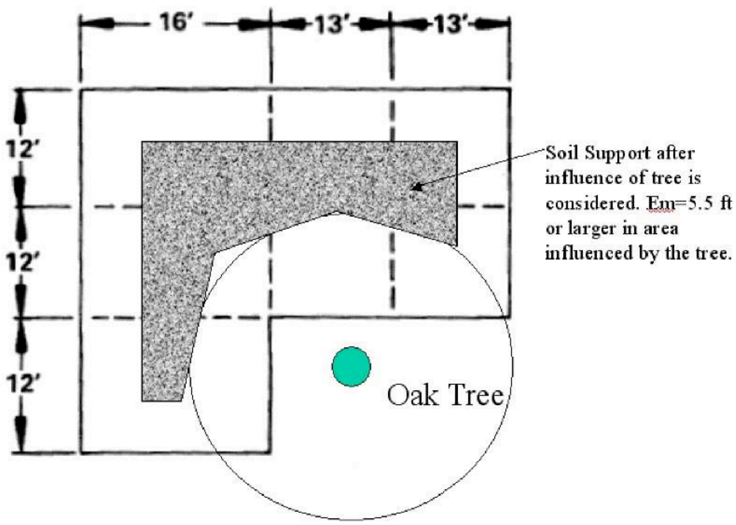

Figure 4.0-5 PTI foundation support model considering the influence of an oak tree.

Such a situation may not be common but it does occur, as the photo below illustrates:

Figure 4.0-6 Trees are commonly left next to a foundation in forested areas.

The calculated deflections for the assumed soil support, due to all the loads stated above, are:

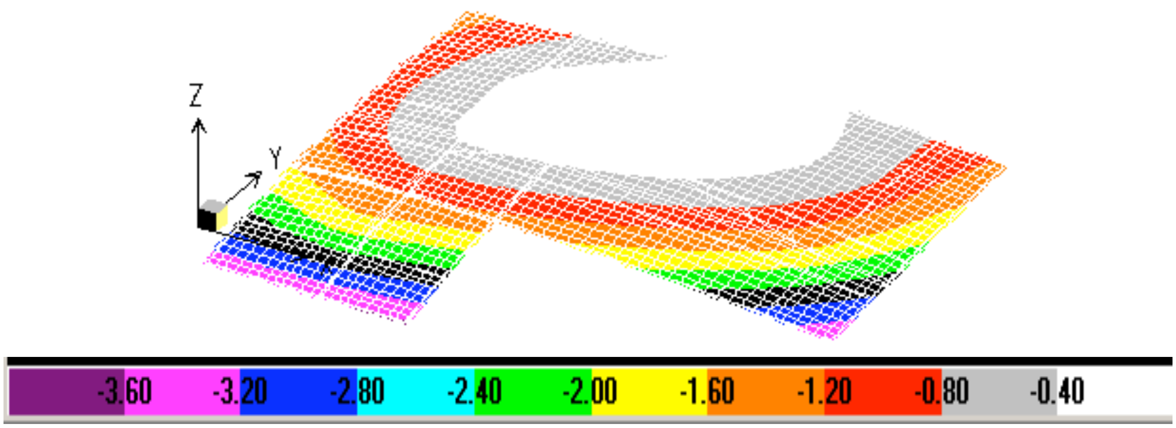

Figure 4.0-7 Computed foundation deformation when a tree is present.
The legend scale is in inches.

The maximum computed differential foundation is 3.77". The presence of the tree has increased the differential deflection over that caused by the dead load, although not as much as could be expected based on studies by Biddle [5]. Biddle's studies indicate that large areas may be affected by trees drawing water out of the ground. This may be because the deflections due to the tree would normally be more significant because the effects of moisture changes on e_m were not fully considered and the effects of moisture changes on y_m were not considered in the calculations.

When trees influence the moisture conditions under the foundation and center lift occurs, the differential deflections from installing the walls and the cantilevered slab weight will be enhanced such that the ACI construction tolerances are exceeded.

PTI Design Procedure, 3rd Edition, does not address at what point during construction that level measurements be taken to satisfy the requirements of ACI 117-90 as well as the TRCC (§304.2 (a) (2)). If the level measurements were taken just prior to the sale of the house, the computed levels for the above example would not meet ACI 117-90 criteria. If the measurements are taken before the walls and the roof are installed, the measurements pass ACI 117-90 and satisfy the TRCC criteria. However, if measurements taken just after foundation placement are used as a baseline to establish foundation deformation caused by plumbing leaks or other causes of foundation distress, there will appear to be more post-construction deflection than actually occurred.

We have demonstrated that the timing of these measurements can make a considerable difference in the measurement results as well as code compliance, and it would be helpful if the PTI Design Procedure addressed when the slab elevations should be taken. If the measurements are taken one day after concrete placement, ACI-117 requirements will be met, but a) the foundation has little deflection due to the superstructure dead loads, and b) the elevations are not useful as a baseline because the slab may be leveled prior to flooring installation and the floor coverings may vary in thickness. If the measurements are taken upon completion of the construction of the superstructure, it may not be possible to follow ACI-117 because the foundation will be covered with flooring surfaces and associated leveling materials. Additionally, the foundation will have deflections due to superstructure dead loads, and it may also have experienced soil/foundation movement induced by soil moisture changes since the foundation was placed.

The FPA recommends that the elevations should be taken as close to the date of completion of the residence as possible. The FPA also recommends that ACI-117 not be specified in the PTI Design Procedure because the addition of floor coverings and normal cyclic level changes during construction may exceed the ACI construction limits.

5.0 IMPORTANCE INDICES FOR SITE PARAMETERS

The homeowner can limit potential foundation distress if the suggestions listed in PTI Design Procedure, 3rd Edition, Section 8.1 (D) DESIGN ASSUMPTIONS, are followed. Some items are under the control of the homeowner (i.e., allowing a tree to grow adjacent to the foundation), others are not (i.e., construction or site preparation deficiencies, paving, tree removal prior to or subsequent to construction, or drought). The PTI needs to address these items.

PTI Design Procedure, 3rd Edition, Section 4.3.3, states that the derivation of the design variable y_m is based upon climate-controlled soil conditions and is invalid when influenced to any significant degree by other conditions. These influences include slopes, cut and fill sections, drainage, time of construction, landscaping, irrigation, trees and dry periods. The last four items are beyond the control of the foundation design engineer.

In PTI Design Procedure, 3rd Edition, Section 4.4.9 states, “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.” In a dry summer many cities restrict water use, thereby forbidding the use of lawn maintenance. Conditions could become so dry that foundations become distressed because watering was restricted for an extended time.

PTI should consider in the design criteria the consequences of the Site Parameters listed in PTI Design Procedure, 3rd Edition, Section 4.4, to foundation performance. Each of the Site Parameters has a different magnitude of influence on the foundation design parameters. The FPA sees as a possibility to introduce an “importance index” to each Site Parameter. These indices would be used by the geotechnical engineer to multiply the e_m and y_m design parameters up to certain maximum values. Such multipliers would be a desirable addition in the design process.

The FPA recommends that the PTI Design Procedure include an “importance index” or multiplier to the PTI design parameters for the Site Parameters listed in PTI Design Procedure, 3rd Edition, Section 4.4.

6.0 DOCUMENT ISSUES

6.1 DOCUMENT FORMAT

The “Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils” [14] and “Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils” [15] do not have the same numbering as that of the PTI 3rd Edition. Therefore, it is difficult to cross-reference these Procedures. Furthermore, the FPA is unsure of how to incorporate the two Standard Requirement documents with PTI Design Procedure, 3rd Edition. In the industry today it appears to the FPA that engineers use only PTI Design Procedure, 3rd Edition, and not the Standard Requirements.

The lack of an index or a searchable PDF copy of the PTI Design Procedure, 3rd Edition, inhibits ease of use of this procedure.

The FPA recommends that the PTI Design Procedure, 3rd Edition:

- *Include an index,*
- *Be prepared in such a way that the chapters, sections, tables and illustrations of this procedure can be cross-referenced with the Standard Requirement documents,*
- *Incorporate the Standard Requirement documents into the procedure, and*
- *Be published as a searchable Adobe PDF digital copy, for easy reference.*

6.2 REFERENCE CORRECTION

The definition of z in PTI Design Procedure, 3rd Edition, Appendix A.1, List of Symbols and Notation, refers to Section 4.5.7. This Section does not apply to z.

The FPA recommends that the PTI Design Procedure, 3rd Edition, correct the Appendix A.1 reference.

6.3 DOCUMENT LIMITATION

Section 6.13.3 in the PTI 3rd Edition refers to Appendix A.8 of the 2nd Edition. It is unreasonable to require the user to obtain a copy of the out-of-print 2nd Edition of the PTI in order to use the 3rd Edition.

The FPA recommends that all prior edition examples and equations referenced in the current edition be reprinted in full in the current edition. As an alternative, the FPA recommends that prior editions of the Design Procedure be available for viewing on the internet.

7.0 CONCLUSIONS

In this commentary paper the FPA has presented areas where improvements to the methods and procedures of the PTI Design Procedure, 3rd Edition, would help to provide more reasonable and responsible engineering designs for residential slab-on-ground foundations. The FPA's recommendations are included in italics at the conclusion of each section. The FPA is of the opinion that the recommended improvements would be beneficial to enhance the performance of foundations designed using the PTI Design Procedure.

The FPA encourages the PTI Committee to consider the recommendations and opinions presented in this paper for its next issue of the PTI Design Procedure.

8.0 REFERENCES

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