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MISCELLANEOUS CONCERNS

SECTION 9A
FORENSIC ENGINEERING

ROBERT WADE BROWN
TOM WITHERSPOON

9A.1 INTRODUCTION

A forensic engineer is the person hired to investigate the damage, deterioration, or collapse of a structure. This duty frequently requires that he or she further "find fault." In some cases the engineer is also asked to prepare appropriate remedial action necessary to make the defendant's structure whole. Refer to "Associates of Engineering Firms Engaged in Geotechnical Engineering" and "The Recommended Practice for Design Professionals Engaged as Experts in Resolution of Construction Disputes" both published by ASFE* in 1993.

The forensic engineer must be an expert in his or her field and have a thorough knowledge and understanding of the engineering problem being considered. This expertise is acquired through both education and specific experience. An engineer should never accept a case unless he or she has the required expertise and can be honest and impartial.

The forensic engineer must be impartial and unbiased as to the cause of the problem, party responsible for the problem, and repair procedures that might be appropriate.

The engineer must arrive at his final conclusion by careful study of the facts (evidence), sound engineering fundamentals (including a thorough literature search), and a careful review of pertinent design calculations. The public should trust the engineer to be the finder of fact.

According to Robert Day, types of visible damage that lends themselves to forensic analysis include:

1. *Architectural damage.* This damage is generally considered as minimal, involving only hairline cracks. Also sometimes referred to as "cosmetic." The maximum values for angular distortion (δ/L) would be less than 1/300 (1″/25 ft).
2. *Functional Damage.* In this category, the differential movement has progressed to the point of adversely influencing the habitability of structure. Outside doors may not latch, interior doors may not close or open. Interior floors are clearly out of level . Leaking roofs or damaged plumbing could also be noted. The maximum values for δ/L could be 1/120 (1″/10 ft).

*American Society of Forensic Engineers.

3. *Structural Damage.* At this level, the safety of the structure may be of concern. Normal foundation repair procedures are usually not effective and upon occasion the structure must be torn down and rebuilt. The maximum δ/L exceeds 1/70 (1″/6 ft).*

Mr. Day is certainly free to present his views for categorizing "forensic" damages. In fact, it seems that several other authors agree, at least in principal. However, we have successfully repaired foundations suffering damage far in excess of the maximum δ/L of 1/70 (1″/6 ft). In fact, repairs to foundations with differential movements in excess of 4″/20 ft (δ/L = 1/60) are fairly routine. Occasionally, a very skillful repair contractor successfully restores foundations suffering differential movement as extreme as 18″ (0.46 m).

The other area of concern to the forensic engineer would be those instances in which the distress is not yet evident. This could involve latent defects or ancillary (monetary) losses. The former refers to "hidden" problems that may be anticipated by testing or engineering calculations. The monetary losses might be incurred from cost overruns or failures to meet deadlines (loss of revenue).

The forensic engineer gathers the data and presents his facts to his client or, as the case may be, to the mediator, arbitrator, or court. This may be done through discovery (interrogatories or deposition), or direct testimony.

In civil disputes, both parties may utilize the services of a forensic engineer. These are referred to as adversarial positions.

In some cases, the engineers for plaintiff and defendant may disagree on fault or amount of ancillary damages. The trier of facts (judge or jury) has the responsibility to resolve this conflict.

When forensic engineers differ on fact, it is incumbent upon each engineer to explain and support his or her position to the trier of fact. Engineering calculations and review of literature should serve as the basis of this proof. There is room for an honest difference of opinion. However, often the differences of opinion are based on bias or ignorance.

The following paragraphs will present the results of forensic studies submitted by 57 engineers practicing in the Dallas–Fort Worth Metroplex. Note, in particular, the *inconsistency* of these engineers' findings and recommendations based on similar problems. As many as half a dozen totally different repair procedures are represented. In most cases, the engineers submitting the report were not working from an adversarial position. That is, each was preparing a structural report to facilitate the sale of property and in many cases only a single engineer was solicited. The following discussion is directed toward lightly loaded structures—homes.

Dealing with a person's home instills, at best, feelings of skepticism and fear. When the public are given totally different solutions to seemingly the same problem, is there any wonder why they might doubt the engineers' competency if not integrity? Why don't all forensic engineers follow the codes of ASFE and/or their local code for professional engineers? This would, hopefully, create a design procedure based on appropriate facts rather than capricious whim.

Table 9A.1 presents a compilation of the recommendations submitted by 57 of the aforementioned practicing engineers. Basically, each report addresses similar foundation repair problems. That is, soil conditions are largely similar and the foundation designs are lightly loaded and generally guided by local specifications. By far and large, these data are not reflective of conditions that are site specific. As a matter of fact, significant geotechnical data and foundation plans are not normally available when engineers are asked to evaluate noncontroversial or less serious potential problems. This service would include "routine" inspections required by lenders, realtors, insurers or municipalities. Disputes involving litigation (insurance settlement, latent defects, defective construction, etc.) are more likely to provide specific geotechnical and design data.

Note that engineers' specifications summarized in Table 9A.1 are quite divergent, although the job conditions are reasonably identical. In some cases, widely different reports were provided by PEs on the *same* foundation. Is there any wonder that the consumers are confused? (All data refer to slab-on-ground foundations.)

*See Robert Day, *Forensic Geotechnical and Foundation Engineering,* McGraw-Hill, New York, 1998.

TABLE 9A.1 Engineers' Pier Specifications

Diameter	>12″		9%
	12″		84.5%
	8″ (Single or Double Shaft)		33.3%
	6″ (Pressed Pile or Cable Lock)		2.6%
	4″ (Steel Minipiles)		5.2%
	Chance Anchors		2.6%
Some engineers offer alternatives. This prevents the sum from totaling 100%.			
Depth	9 ft		11.7%
	10 ft		11.7%
	12 ft		61.2%
	15 ft		11.7%
	>15 ft		2.9%
Rebar	2 #3's (8″ or 12″ diameter)		27.6%
	3 #3's (8″ or 12″ diameter)		17.2%
	4 #3's (12″ diameter)		6.9%
	2 #4's (12″ diameter)		3.4%
	3 #4's (12″ diameter)		6.9%
	4 #4's (12″ diameter)		31.0%
	4 #5's, caged (12″ diameter)		13.8%
	> #5's (12″ diameter)		6.9%
Belled (12″ diameter)			25.6%
OC	12″ drilled concrete	5′–6′	30.0%
		7′–9′	67.0%
		> 9′	3.0%
	8″ drilled concrete	5′–6′	67.0%
		7′	25.0%
		8′	8.0%
	6″ pressed pile (cablelock)	5′–6′	100.0%*
	4″ minipile (chance anchor)	5′–6′	100.0%*
Mudjacking			80.5%
Interior piers (slab foundation)			<2.0%

*Too few reports included to be statistically significant.

9A.2 ENGINEERS' SPECIFICATIONS

It is understandable to have several different repair approaches to the same problem. While this might be a little confusing to the consumer, at least initially, it is something that can be mostly understood. After all, foundation repair in itself is a matter of compromise. It is another matter when the engineers' specifications introduce a wide disparity in the design of the same underpin. For example, refer to the various "specifications" suggested for the 12″ diameter drilled pier.

The decreasing order for the engineers' selection of the various methods for repair is as follows: 12″ diameter drilled pier, 84.5%; 8″ drilled pier (single or double shaft), 33.3%; 4″ steel minipiles, 5.2%; 6″ pressed pile or cable lock, 2.6%; and the helical screw anchor, 2.6%. Other repair options, such as polyurethane injection, soil stabilization using sulfuric acid or enzyme base chemicals, pressure lime injection, and hydropiers were either recommended by less than 2% of the engineers or not at all.

TABLE 9A.2 Soil Bearing Capacity Versus Underpinning Design

Assumptions:

Structural load on perimeter beam

Frame Single Story	400 plf
Brick Veneer Single Story	600 plf
Brick Veneer Two Story	800 plf

Foundation construction

Beam: 24″ deep (20″ net) × 10″ wide

2 #4's top and bottom (3″ cover).

Slab: 4″ thick

#3's 18″ OCBW

Soil Characteristics

γ = 120#/ft3

q_u = 2tsf (1 ft), 2.5 tsf (4 ft), 3.0 tsf (12 ft)

PI > 15

$\sigma_h = k_v$, $k = 1.0$

Safety Factor Desired (refer also to Appendix 9A.A) 3.0

Underpin			Soil Capacity	
1. Drill Pier				
12″ dia. to 12′	Q_{BEAM}	= 23,324 lb	M_{LOAD}	= .6 ft × 4800 lb = 2800 ft-lb
8 ft OC with	Q_{HAUNCH}	= 21,080	M_{RESIST}	= $e \times k/2(h)^2\gamma \times 1$ ft
5 ft^2 haunch	Q_{TIP}	= 4,710		= 6 ft × (144)120/2 × 1 ft
e = 7″ = 0.6 ft	$Q_{FRICTION}$	= 8,054		= 51,840.00 ft-lb
	Q_{SOIL}	= <u>57,168 lb</u>		
	Q_L	= 8 ft × 600 plf		
		= <u>4,800 lb</u>		
	SF = 12.0		SF = 18.5	
2. Pressed Pile (cable lock)				
6″ diameter	Q_{BEAM}	= 18,333 lb	M_{LOAD}	= 0
	Q_{HAUNCH}	= NA	M_{RESIST}	= $e \times k/2\ (h^2)(\gamma)$
6 ft OC to 12 ft	Q_{TIP}	= 1,176		= (7) (144/2)120 × .5 ft
e = 0	$Q_{FRICTION}$	= zero		= 30,024 ft-lb
	Q_{SOIL}	= <u>19,509</u>		
	Q_L	= 6 ft × 600 plf		
		= <u>3600 lb</u>		
	SF = 5.4		NA	

Note: 1) Unless the q_u is increased due to compression caused by driving, the house *will not* raise.
2) The load capacity is carried by the perimeter beam. The pile is incidental.
3) Unless the bearing capacity afforded by the perimeter beam is restored, the pier *will not* sustain the load.
4) Increase the spacing much beyond 6 ft could be catastrophic.
5) $Q_{FRICTION}$ is a nonfactor since, elements of the pier are not connected. Friction between the blocks due to compressive load is only factor holding the string of pressed cylinders together.

3. Minipile (hydraulically driven)				
3.5″ OD to 12 ft	Q_{BEAM}	= 16,667 lb	M_{LOAD}	= .67 ft × 3000 = 2010 ft-lb
		Figure M_r on first, 4 ft long joint		
5 ft OC	Q_{HAUNCH}	= NA	M_{RESIST}	= $e\ (1/2\ \gamma h^2) \times A$

(continued)

TABLE 9A.2 *(continued)*

$e = 8'' = 0.67$ ft	Q_{TIP} = 402 lb $Q_{FRICTION}$ = 1,573 lb Q_{SOIL} = $\underline{18{,}642 \text{ lb}}$ Q_L = 5 ft × 600 plf = $\underline{3000 \text{ lb}}$		= 2.6 × [120 × 16]/2 × 0.3 ft = 749 ft-lb
	SF = 6.2	SF = 0.6	
	∴ pipe will deviate		

Note: 1) Unless the beam-bearing capacity is restored, pile will fail.
2) Some questions persists concerning the advisability of considering skin friction as a viable load carrying factor.
3) The load capacity provided by the pier is incidental though slightly superior to the pressed pile.
4) Superloading a pile (usually accomplished by driving a single pier at a time), will result in increased applied movement and load of at least two or three fold. The indicated factors of safety will be reduced accordingly. Increasing the spacing will not be the same result.

Authors' comment: Aside from the obvious concerns of most engineers, the authors are finding serious problems related to rotational shear. Many instances of horizontal shear of the perimeter beam have been documented. It should be noted that a high percentage of the failures occurred when the contractor exceeded the spacing of 5 ft OC.

The spreadfooting is conspicuous by its absence. The "old standby" has not lost its following but economics have dampened its use. The spreadfooting costs more than a 12″ drilled pier while its performance is equivalent. In making the choice of

method, the repair company or engineers should provide the consumer proof of the effectiveness of the method they propose. Refer to Table 9A.2.

The point most confusing to the consumer is again when various engineers propose widely different specifications for the same method. Review Table 9A.1. This raises serious questions as to the competence, reliability, and/or voracity of the engineering community as a whole.

The following "Summary of Table 9A.1" outlines the "consensus" of opinion among the reporting engineers.

9A.3 SUMMARY OF TABLE 9A.1

1. *Pier Diameter.* The consensus suggests the use of 12″ diameter drilled shafts by 84.5%. This coincides with data published by Greenfield and Shen (*Foundation in Problem Soils,* Prentice Hall, 1992). The disturbing fact is that 15.5% suggested other designs that probably would not pass a reasonable mathematical analysis.
2. *Pier Depths.* The *consensus* for "proper" pier depth seems to be 10–12 ft (72.9%). Greater depths are readily available at additional costs, reportedly as much as \$30 to \$50 per foot. In normal cases, this added depth affords little benefit. Site specific geotechnical data could introduce an exception.
3. *Reinforcing.* Reinforcing runs the gamut from a single #3 to a single #9, with multiple bars of various diameter in between. The design and placement of steel depends largely upon the design concern. For example, a single rebar might provide adequate resistance to tensile stress but provides little benefit against lateral stress or bending movements. Multiple bars are required for the

latter. For normal applications, 4 #3's or 4 #4's are the general preference for 12″ diameter piers (37.9%) and 2 #3's or 3 #3's for 8″ diameter shafts (100.0%).

4. *Belled Shafts.* Most engineers (74.4%) do not specify that the piers be belled. Fortunately, expansive soils are generally blessed with a good bearing capacity (q_u). Hence, belling is not required to provide the greater load capacity. If proper precautions are taken to avoid access to water at pier depth or along the pier shaft, belling is not required to prevent heave; hence, belling is frequently not required. [Belling is generally available for the added cost of $50 to $100 (12″ shaft) per pier.]
5. *On-Centers.* Pier spacing also varies not only with pier diameter but between engineers on the same diameter. The general opinion for various diameter piers however suggests: 8 ft OC (12″ diameter), 6 ft (8″ diameter), 5–6 ft OC (6″ diameter), and 5–6 ft OC (4″ minipile).
6. *Mudjacking.* Eighty one percent of the engineers recommend mudjacking slab foundations. This is fortunate, since residential slab foundations are not designed (or intended) to be bridging members; hence, the name "slab-on-ground." Mudjacking restores the soil bearing required by the slabs' design.
7. *Underpinning Interior Slab Floor.* Only about 2% of the engineers constituting the study recommended the use of piers to support interior floors. Breaking out the slab to install piers is not only unwise but extremely destructive and expensive. Over the past 35 years, mudjacking has provided acceptable performance in restoring interior slabs to proper grade. One noteable exception might be the need to raise a heavily loaded interior fireplace. Another exception might be that occasion where it becomes necessary to raise a common firewall on a multiple story apartment building. Mudjacking alone would not likely provide the desired raise in either of these cases but would be required to fill voids. Refer also to Section 7B4.4.5.
8. *Alternatives,* About 28% of the engineers offered alternatives. Generally, the alternatives were between the use of 12″ diameter piers and 8″ diameter piers, single or double shaft. Some 5.7% of the total engineers sampled offered the following *alternatives:* (a) 12″ diameter, 6 ft OC, 2 × bell, with 4 #5's caged to 12 ft, *or* (b) 8″ diameter, 6 ft OC, no bell, with 3 #3's to 12 ft. It would seem that "alternative" equates to "equivalence." Who in their right mind would ever consider (b) to be equivalent to (a)? Note particularly the spacing and reinforcing. Thankfully, none of the engineers making this recommendation were either civil or structural engineers.

9A.4 BEARING CAPACITY OF VARIOUS UNDERPINS

Table 9A.2 presents compararisons for soil bearing capacity versus various underpinning options. It is important to note the assumptions. For best use, actual values should be determined where possible. (Among the three methods, 12″ diameter drilled piers, 6″ press pile, and 3 1/2″ steel minipile, only the 12″ diameter pier passes the scrutiny.) Figure 9A.1 depicts the mechanics involved with the example calculations.

Note: When dealing with the 12″ diameter drilled piers, the moments (M_L and M_R) do not come into play until the beam load is actually transferred to the shaft during raising. Conversely, these moments are active concerns with the hydraulic driven steel minipiles when the very first joint is driven and then intensifies with each additional section of driven pile. The movement calculations shown in Table 9A.2 and Figure 9A.1 are simplified but serve the intended purpose.

9A.5 CONCLUSIONS

Probably the most serious affront attributed to the engineering community lies with those few individuals (estimated at less than 5%) who are largely or totally funded by the insurance companies.

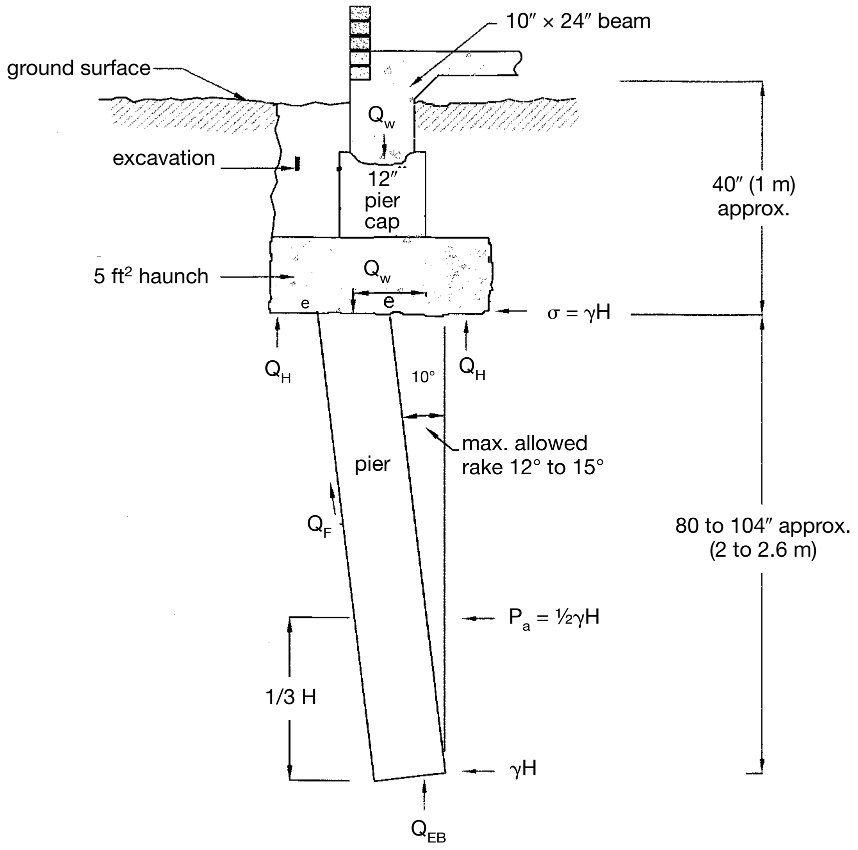
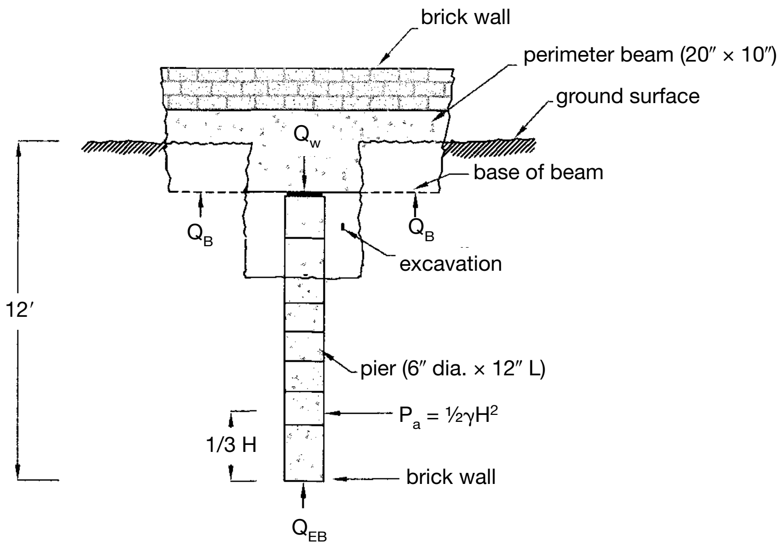


FIGURE 9A.1 Load capacity of selected underpins.

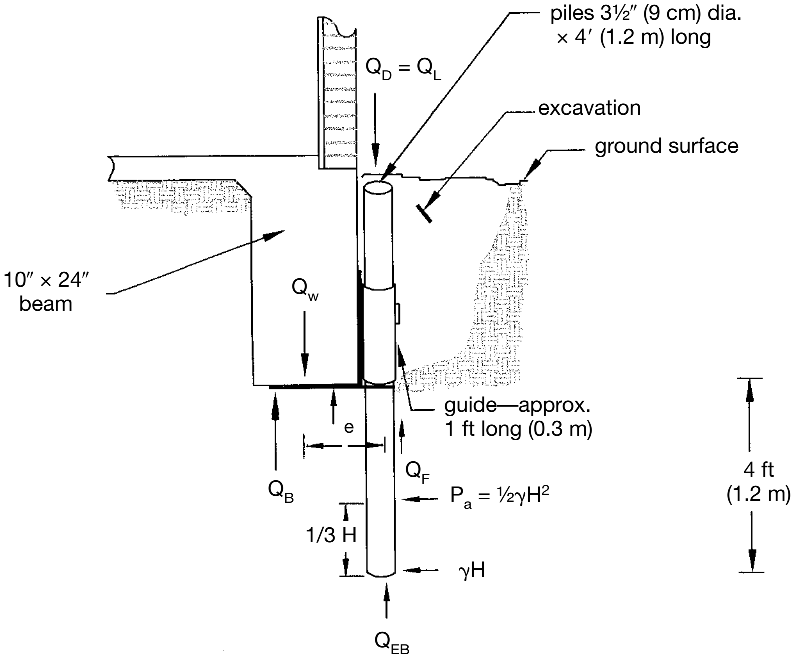
Aside form the aforementioned dilemma in dealing with repair methods, these engineers find, almost without exception, that sewer leaks never causes foundation problems. (Heave is a casualty loss generally covered by the home owners insurance.) Rather, they tend to blame tree roots or "natural doming." (Both of which are largely discounted by recent research.). If either of these were the cause of apparent upheaval, why does the problem generally abate when the source of water is eliminated? (Sewer repairs are the most preponderant solution.) Seldom does the engineer establish the in-site soil moisture either *prior* to or *after* the leaks. The aftereffect is difficult to remedy, since the location of eventual moisture accumulation is not easy to predict. If insitu moisture content approaches or exceeds the PL, little or no soil swell (upheaval) might be anticipated. For a detailed study of the foregoing and other contoversial areas, refer to Tables 9A.3 and 9A.4.

Floor elevations are often used to suggest that settlement, apposed to upheaval, is the preponderant cause of the distress. The high point is often used as the zero. Obviously, any other elevation is



(B)

PRESSED PILE



(C)

ECCENTRICALLY DRIVEN PILES

FIGURE 9A.1 *(continued)*

then going to show as a negative reading. This generally promotes the interpretation of perimeter settlement. A single set of elevations can also do more harm than good. The elevations show the *current* contour of the slab but provide no way of determining differentials relative to initial construction. Seldom, if ever, is it wise to "level" a foundation based on elevations alone.

Engineering errors in judgement do not generally prevail over the long haul. However, at some point, this situation does cause serious distress and often substantial costs to the public (home owners). A good guess would be that the defendant insurance companies eventually are

forced to cover losses in well over 50% of the cases taken to litigation. The bottom line is simple. If the engineers were truthful at the start, the expense, inconvenience, and stress of litigation might be spared both parties, defendant and plaintiff alike. Estimates for attorney fees charged to *each* party are in excess of $25,000. One would think that in view of court opinions such as Nicolau vs. State Farm, Corpus Christi Appeals Court 869 SW 2nd 543, the insurance companies and their engineers would take note and clean up their acts.

The engineering community needs to put forth a unified, honest, capable, and dependable front if we are to gain and maintain the respect of the public.

A last thought: An Engineer serving as an expert witness will never be any better than the attorney who hires him. Court procedures controlling expert testimony require, in general, that the expert merely answer questions. If the attorney does not ask the proper questions, the expert witness cannot bare the full truth before the court.

Table 9A.3 Easy References for Controversial Topics

I. Chen *Foundation on Expansive Soils,* Elsevier, 1988.

Settlement: "Theoretically possible but little evidence of downward movement beneath covered structures exists." (p. 103) "The end result of shrinking beneath a covered area seldom causes any structural damage and is not an important item to soil engineers." (p. 102)

Slab fill versus water movement: "Water from a sewer leak can travel without resistance throughout the gravel (fill) bed and saturate the entire slab area." (pp. 202–203)

Moisture barrier: "Thermal osmosis can introduce additional moisture through a barrier." (p. 229) "It is doubtful that the installation is of sufficient benefit to warrant the cost." (p. 235) "To date the merits of these installations on a long term basis have not been verified." (p. 234)

Pier diameter: "To maximize dead load pressure, pier diameter should be min and span max." (p. 151) "Pier holes less than 12″ diameter are difficult to clean as well as pour and reinforce." (p. 151)

Pier heave: "Pier movement is unlikely unless associated with perched water. Pointing a finger to inadequate void space or slight mushrooming is not usually justified." (p. 337) "Exclude top 5 ft from bearing capacity calculation." (p. 147)

Pier mushroom: "In most cases, the portion of pier uplifting force that can be assigned to the mushroom effect is actually quite small." (pp. 156–157) Refers to "pier mushroom" effect often observed at surface end of a poured concrete pier.

Lime stabilization: "The success of lime treated subgrade is doubtful." (p. 271) "Lime treatment of sulphate-bearing clay can induce expansive reaction (ettringite)." (p. 275) "Large quantities of water present potential danger of triggering an excessive amount of swell in deep seated soils." (p. 278)

Prewetting: "Highly questionable if prewetting can produce a uniform moisture content." (p. 259) "It is doubtful if prewetting is an important construction tool for building foundation on expansive soils." (p. 259) "In covered areas the moisture content of the covered soil seldom decreases." (p. 258)

Trees: "It is doubtful whether large tree roots will pose a problem in high swelling soils." (p. 248)

Spreadfooting versus Pier: "The use of a short pier is no different or more desirable than an individual pad." (p. 152)

(continued)

Table 9A.3 *(continued)*

II. Greenfield and Shen, *Foundation in Problem Soils,* Prentice Hall, 1992.

Pier diameter: "The optimum pier diameter for residential use is 10″ to 12″ diameter. Pier diameter should be operable minimum with max spacing for max dead load." (pp. 50, 58, 74, 78, 80)
Watering (prewetting): "This method has been ineffective." (p. 65)
Pressure lime injection: "The long term effectiveness has not been proven." (p. 66)
Void boxes: "Void boxes deteriorate to create openings for water flow as well as access to rodents and insects." (p. 81)

III. Holland, Laurence and Crimino "The Behavior and Design of Housing Slabs on Expansive Clays," *Expansive Soils,* 4th International Conference on Expansive Soils, Denver Co, 1980. (Perimeter and interior beam depths, 12.″)

Center Doming: "*Theoretically,* with time the heave under a slab will slowly progress inward until a center heave or mound distortion mode will form under the slab (Figure 1e). There is much observation and research evidence to suggest many housing slabs may not finally develop the center heave distortion mode." (pp. 449–450) [Acknowledges moisture increase in confined soil but alludes to a more uniform distribution of water as opposed to a center heave.]

"When slabs are placed on poorly draining, heavily fissured, dry clay sites, it is feasible that sufficient water may enter beneath the slab quickly enough to allow the development of a center heave situation. This condition was believed to be responsible for the single center doming noted in the study of 21 foundations." (p. 457)

IV. Holland and Laurence, "Seasonal Heave of Australian Clay Soils," *Expansive Soils,* 4th International Conference on Expansive Soils, Denver CO, 1980.

Center doming: "It appears that at most developed sites, the center heave mode may not form in the life of the structure." (p. 313)

V. William and Donaldson, "Building on Expansive Soil in South Africa." *Expansive Soils,* 4th International Conference on Expansive Soils, Denver CO, 1980.

Pressure lime injection: "A limited trial with lime injection was unsuccessful and no further work has been undertaken in this diversion." (p. 842)

VI. Jones and Jones, "Treating Expansive Soils," *Civil Engineer,* August 1997.

Lime slurry pressure injection: "Neither the lime nor the water penetrates deeply enough throughout the soil, so a uniformly treated soil mass is not produced." (p. 65)

VII. Goode, Hamberg and Nelson, "Moisture Content and Heave Beneath Slabs on Grade," *Fifth International Conference on Expansive Soils,* Adelaide, Australia, 1984.

This paper compares soil moisture variations recorded at intervals of 1 year, 1½ years, and 2 years. The "real" foundations consisted of a perimeter beam on piers with a floating slab floor (basement). The "simulated" foundations consisted of an impermeable membrane (polyethylene) placed on the ground surface after stripping the vegetation. These surface barriers were covered with a 5 cm (2 in) layer of sand and a 10 cm (4 in) layer of gravel to protect the barrier and simulate the weight of a concrete slab. Vertical vapor barriers were installed on two of the four test sites to a depth of 2.5 m (8 ft).

Center dome: "As would be anticipated, the soil moisture content beneath both the basement floors and the "simulated" foundations increased with time. However, in no instance did the moisture profile even closely resemble a "doming" contour." (pp. 214–216)
Vertical moisture: "The vertical moisture barriers were ineffective in reducing total heave at the test plots. However, the barriers were effective in delaying the *rate* of moisture and heave." (p. 216)
Lime treatment: "Treatment of the subgrade soils with hydrated lime (mechanically mixed into top 30 cm [12 in] of base) does not appear to be effective at this site. (p. 216)

(continued)

Table 9A.3 *(continued)*

VIII. Nelson and Miller, *Expansive Soils,* Wiley, New York, 1992.

Moisture barriers: "The results indicated that as for horizontal barriers, increases in water content still occurred and the amount of the heave was not decreased. However, the heave was more uniform for slabs with vertical barriers than for slabs without." (pp. 161 and 205)

IX. Tucker and Poor "A study of Behavior of Slab Founded on Active Clay Soils." Report TR-5-73, Construction Research Center, UNA, November 8, 1973. Published in part as, "Field Study of Moisture Effects of Slab Movements," *Journal of Geotechnical Engineering,* April 1978.

Fifty nine of 69 foundations included in this study were FHA type B with 16″ × 18″ (40 × 20 cm) perimeter beams, the base of which was generally less than 6″ (15 cm) below grade. After October 1, 1963 the foundation design was altered by principally increasingthe perimeter beam to 20″ × 10″ (50 cm × 25 cm) and adding stiffer beams 20″ × 8″ (50 × 20 cm) 16 ft (5 m) on centers both ways. However, once again the beam depth was held to about 6″ (15 cm) (pp. 24–26). With beams as shallow as those constituting this study, the net result is almost like addressing the behavior of concrete flatwork.

Generally the variation between the first and second sets of slab elevation are insignificant [0.02 ft to 0.04 ft (0.24 to 0.48 in).] There were a few exceptions with one measurement of 0.18 ft (2¹/₈″) and three of 0.12 ft (1.4″). (pp. 41–42) The facts of concern include:

1) The slabs were unattended and totally at the mercy of the elements.
2) There could be no influence due to sewer leaks since the properties were uninhabited. However, drainage problems' could be persistent if water were available.
3) This observation could handily be the relationship of both shallow beams and ambient conditions of evapotranspiration.

The soaker tests seem to be a little inconsistent. Soaking the south perimeter (8-11-72) created a heave of 0.6″ in the south central slab area. (Wonder where the sewer line runs?) Soaking the northeast perimeter (8-13-72) increase the heave area. Continued soaking at the northeast corner (8-15-72) reduced the heave in the south central area by 1.2.″ (pp. 114–118)

If moisture increases beneath the foundation are a result of "normal" flow, how could "tension cracks" explain the water migration? Tension cracks would not be a likely occurence beneath a slab foundation.

No doubt—maintaining soil moisture will abate foundation movement.

Tree Roots:

1) All roots described by the author were within 6″ (15cm) from the surface. (pp. 61, 76, 179) In all likelihood the trees preexisted the placement of foundations.
2) The canopy width of a tree is reflective of its root spread. Height plays no part in this relationship. In 1973 this was not a commonly recognized fact.
3) Combining the authors' data from pages 37, 41–44, 62–72 it appears that only 10 (14.5%) of the foundations had trees in a proximity where the canopy could reach or overhang the perimeter. Of the ten the authors' had rated 8 as having "serious damage." Overall, these 8 represented (27.6%) of the so-called "severely damaged.

There is absolutely no evidence that would suggest that tree roots were responsible for the cited 10 failures. In fact the authors state "The result of the tree survey and differential movement data indicate that movements of slabs along S.H. 360 *may in part* be due to dessication of the soils by tree roots. (p. 136)

Natural Doming:

4) "Moisture migrates from wet to dry." (pp. 144, 181)

It would seem that this alone would preclude any so-called natural "center doming." Water would flow from a wet area (center or otherwise) to the dryer area (perimeter).

(continued)

Table 9A.3 *(continued)*

This lateral water flow would be facilitated by the much higher horizontal as apposed to vertical permeability ($K_H = 10\ K_V$). The shallow beam and total lack of maintenance noted in this study would definitely promote edge or perimeter moisture loss.

5) "Moisture contents are not always higher at the center. . . ." (p. 145)

This seems to be an understatement, assuming natural conditions, the interior soils will tend to be wetter than at the perimeter. The soil moisture profile will, however, move closely resemble a plateau or mesa, as opposed to a dome. A "doming" profile is often noted in conjunction with water accumulation beneath the slabs, particularly as a result of sewer leaks.

6) How many of the foundations had heaved due to water accumulation (bad drainage and/or utility leaks) beneath the slab prior to the 1971–1972 study? A layout for the sewer lines might have supported a different conclusion as to the cause of the movement.
7) In the 1973 the full impact of utility leaks (particularly those sewers related) on foundation stability had not been fully recognized. The authors did however note this as a concern. (pp. 9, 28)
8) The survey of *newly constructed* slab foundations indicated central high spots of to about 1.44″ (0.14 ft). (pp. 44–47)
9) "Below about 5 ft (1.5m) the soil moisture content does not vary more than 1%." (p. 154)

If soil moisture remains constant, there is not differential movement. Most foundations are designed to resist moisture differentials of 1 to 3%.

10) "The swelling decreases as the overburden increases." (p. 179)

Insignificant soil swell is noted at depths of about 5 ft. Assuming a unit weight of 120 pcf, the overburden pressure at 5 ft depth would be 600 psf. Then would it follow that a structural load on the perimeter beam of 600 psf would eliminate or minimize beam heave? [The lightly loaded interior slab (50 psf) and stiffener beams (175 psf) would both be more prone to heave.] Actually, the soil stability at or near 5 ft is due more to the influence of SAZ than overburden. However, it is known that overburden reduces soil swell.

There is no compelling data provided by this study that would support the so-called "natural center doming." Without a doubt, soil moisture tend to increase (accumulate) beneath a slab foundation. The moisture will develop higher values slightly inside the perimeter. Interior areas along the perimeter (and particularly at corners) will be dryer.

X. R. W. Brown, *Foundation Behavior and Repair,* 3e, McGraw-Hill, New York, 1997.

Tree Roots:

1) "The detrimental effects on foundations from transpiration (roots) appear to be grossly overstated." (pp. 14–28)
2) If the perimeter beam is at least 18 in (0.45 m) deep, intruding roots are not likely to cause any serious concern with respect to interior foundation areas." (p. 147)
3) "If trees pose the problems which some seem to believe, why do not *all* foundations with like trees in close proximity suffer the same relative distress? In literally thousands of instances where foundation repairs are made without removal of trees, why do not the foundation problems recur, at least sometimes?" (pp. 274–278)

Upheaval:

4) "In expansive soils, slab heave results almost without exception from the introduction of moisture beneath the foundation." (pp. 151–158)
5) "It is interesting to note that in most cases foundation movement ceases shortly after the source of water has been eliminated." (p. 155)
6) "The daily input of water (a leak required to produce a 4% increase) would be only 143 drops per day (0.1 gal/month over 12 months)." (pp. 161–166)

(continued)

Table 9A.3 *(continued)*

Upheaval is the cause of 70% of all slab repairs in the Dallas–Fort Worth Metroplex. Utility leaks are responsible for a majority of these failures. The so-called "natural center doming" has not been identified in over 30,000 repairs.

Moisture Barrier:

7) "A horizontal barrier does little to prevent or lessen the moisture build-up beneath a foundation over a time span of several years." (p. 262)
8) "Vertical barriers tend to reduce first year or so; however, after 4 to 5 years the results become nearly the same with or without the barrier." (p. 262)

Lime Slurry Pressure Injection (LSPI)

9) "LSPI unfortunately does not seen to be an effective measure for foundation repair." (p. 256)

Settlement:

10) "30 % settlement compared to 70% upheaval."

In this study of Dallas–Fort Worth foundations performed over 35 years, "settlement" included such failure as erosion, consolidation, and compaction.

XI. Popescu, "Engineering Problems Associated with Expansive and Collapsible Soil Behavior," *7th International Conference on Expansive Soils,* Dallas, TX, August 1992. Volume 2.

Tree Removal: "The most damaging situation is where trees have been removed from positions either adjacent to existing buildings or adjacent or beneath the locations of new buildings." (p. 26)

XII. Edil and Alamazy, "Lateral Swelling Pressures," *7th International Conference on Expansive Soils,* Dallas Texas, August 1992.

Lateral Pressure versus Moisture Content: "The lateral pressure also decreases with higher moisture content; however, not as much as does the vertical swelling pressure." "As the initial moisture content increases, the lateral swelling pressure decreases." (p. 230)

Lateral Pressures versus Surcharge Load: "The surcharge pressure reduces the percent swell as it increases. But when the surcharge pressure is higher, the lateral pressure is also higher. The lateral pressure develops as a result of both the swelling tendency and the lateral deformation tendency in response to the applied vertical pressure. If the vertical pressure is increased it will restrain swelling in the vertical direction and increase the potential for volume expansion in the lateral direction. These two mechanisms together, cause a substantial increase in the lateral pressure when the vertical pressure increases. (p. 231)

XIII. Day, Forensic Geotechnical and Foundation Engineering, McGraw-Hill, New York, 1998.

Root Heave: "Damage to structure caused by tree roots is very common. Damage usually occurs to lightly loaded structures such as sidewalks, patios, roads and block walls, where the physical increase in size of growing roots causes uplift and differential movement." (p. 247)

XIV. Struzyk and Newton, "How Trees Affect Slab-on-Grade Foundations," ASCE Section meeting, San Antonio, TX, September 1996.

Tree Roots versus Foundation Stability: "At Case 1, there was no distinct down-slope pattern; however, there were large trees on all sides of the residence. There was a definite downward slope at the east side of Case II. In Case II the tree roots did not appear to extend under the foundation because the perimeter beam acted as a root barrier." (pp. 149–150)

In both cases the foundations were surrounded on four sides by mature trees. (Eight Oak trees in Case I and Nine Pear, Oak, Elm, Ash, and Hackberry trees for Case II.)

XV. *Construction and Maintenance Procedures Manual for Post Tension Slab on Grade Construction,* 2nd Edition. Post Tension Institute, 1717 W. Northern Ave. #144, Pheonix, AZ 85021

(continued)

Table 9A.3 *(continued)*

Trees:

1) Tree removal can be safely accomplished provided that the tree is no older than any part of the house, since the subsequent heave can only return the foundation to its original position." (p. 67)
2) "When a tree is older than the foundation it is not considered advisable to remove the tree because of the danger of inducing damaging heave, unless the foundation was designed for the total computed vertical movement." (p. 67)

Intrusive Damage to Slabs:

1) Property owners should also be made aware of the precautions that are to be taken when modifying or cutting holes in foundation slabs (p. 65)

XVI. R. W. Brown, *Foundation Repair Manual,* McGraw Hill, New York, 1999

Sewer Leaks versus Heave:

"Another curious point lies in the facts that (1) the group of engineers (who basically state that sewer leaks are not a significant cause of foundation distress) invariably recommend that the leaks be repaired, and (2) once the leaks are repaired, foundation movement generally ceases after some relatively short period of time. (The exception is most often the result of another *undetected* leak.) Both facts clearly suggest that the sewer leak is, in fact, the source of the problem." (p. 8.13)

This reference offers several pages of pertinent information dealing with sewer leak problems. (pp. 8.12 to 8.14

Another source, *Foundation Behavior and Repair,* 3rd Edition, also offers excellent information (pp. 151–158 and 161–166).

From the literally hundreds of engineering reports reviewed by the authors, the only engineers who tend to question, deny, or deemphasize the relationship between slab heave and sewer leaks seem to be only those individuals hired by the insurance companies.

Tree Roots: "Roots per se provide a benefit to soil (and foundation) stability since their presence increases the soils' resistance to shear." (p. 20; also see pp. 14–18)

XVII. Mark Peterson, "Expert: Trees not 'root cause' of foundation damage in area." *San Antonio Express,* Views, August 28, 1996.

Trees: "Trees are not major causes of foundation instability."

XVIII. T. J. Freeman et. al., *Has Your House Got Cracks?* Institute of Civil Engineers, London, 1994.

Trees: "It follows that large trees should be left in place where ever possible." (p. 112)

Tree Root Prunning: "By cutting through tree roots, they inevitably upset the equilibrium in the soil even if no trees are removed; this in turn generates lateral movements in the soil, which tend to push the foundation sideways." (p. 112)

XIX. T. H. Wu et. al., "Study of Soil–Root Interaction." *JGE,* vol. 114, December 1998.

XX. J. Choppin and I. G. Richards, *Use of Vegetation in Civil Engineering.* Butterworth, London, 1990.

Authors' comments: The presence of structural tree roots beneath a slab foundation are more prone to enhance stability than to exacerbate settlement. Refer also to Sections 1A and 7A, this volume. Structural roots would tend to compress and strengthen the soil. Also, the structural roots account for minimal soil moisture loss. If these roots have any effect on the foundation, the net result would more logically lean toward upheaval. This opinion has been suggested by a number of published authors including T. H. Wu, N. J. Choppin, Fau Chen, Robert Wade Brown, Robert Day, and Mark Peterson.

XXI. Chein Fu, Court Testimony, Nicolau vs. State Farm Insurance, No. 13-92-467-CU, Court of Appeals, Corpus Christi, TX, December 16, 1993.

(continued)

Table 9A.3 *(continued)*

Sewer Leaks versus Flow: "Water tends to follow plumbing ditches. Ultimately the water may expand outward to great distances through the layer of cushion sand between the clay substrata and foundation."

XXII. Samuel E. French, *Design of Shallow Foundations,* ASCE Press, 1991.

Center Doming: "For many years it was a belief that when a desiccated clay was covered by a building, the evaporation of the pore water at the surface would be drastically reduced. The water content would therefore increases sharply, contributing to the swelling of the clay. More recent research has shown that there is no continuous rise of ground water (soil moisture) in a desiccated clay. The existence of desiccation fissures thoroughly and efficiently disrupts this phenomenon." (p. 330)

Table 9A.4 Cross References for Table 9A.3

A. Central Slab Natural Doming:
- III. Holland, Laurence, and Crimino
- IV. Holland and Laurence
- VII. Goode, Gamberg, and Nelson
- IX. Tucker and Poor, (4) through (10)
- XXII. Samuel French

B. Lime Stabilization—Pressure Lime Injection:
- I. Chen
- II. Greenfield and Shen
- V. William and Donaldson
- VI. Jones and Jones
- VII. Goode, Hamberg and Nelson
- X. Brown, (9)

C. Moisture Barrier:
- I. Chen
- VII. Goode, Hamberg, and Nelson
- VIII. Nelson and Miller
- X. Brown, (7), (8)

D. Lateral Pressure:
- 1. Versus Moisture Content
  - XII. Edil and Alamazy
- 2. Versus Surcharge Load
  - XII. Edil and Alamazy

E. Piers:
- 1. Diameter
  - I. Chen
  - II. Greenfield and Shen
- 2. Pier Heave (mushrooming)
  - I. Chen

F. Prewetting:
- I. Chen
- II. Greenfield and Shen

G. Settlement (foundation):
- I. Chen

(continued)

Table 9A.4 *(continued)*

X. Brown
XVI. Brown
H. Spreadfooting versus Pier:
I. Chen
I. Sewer Leaks versus Slab Foundation:
1. Water migration through fill material beneath slab foundation
Fill Material
I. Chen
XXI. Fu
Sewer Lines
X. Brown, (4) through (6)
XVI. Brown
J. Tree Impact on Foundation Stability:
1. Effective of Roots on
I. Chen
IX. Tucker and Poor (1) through (6)
X. Brown, (1) through (3)
XIII. Day (heave)
XIV. Struzyk and Newton
XVII. Peterson .
2. Tree Removal
XI. Popescu
XV. PTI Manual, 2nd Edition
XVIII. Freeman, et. al.
3. Prunning Roots
XVIII. Freeman, et. al.
K. Upheaval (see also Sewer Leaks):
X. Brown, 4) through 6)
XVI. Brown
L.Void Boxes:
II. Grenfield and Shen

9A.6 APPENDIX

The following table will provide the building weights necessary to calculate structural loads. Calculated values can be used rather than those assumed in Table 9A.2

TABLE 9A.A Structural Weights for Concrete Foundations (Q_w)

1. Interior Floors:	
4″ Concrete Slab	50 psf
(Add or subtract 12.5 psf per inch thickness)	
Wood (flooring, subflooring on 2″ × 10″, 16 ft OC)	7 psf
2. Concrete Interior X-Beams (10″ × 16″)	165 plf

(continued)

TABLE 9A.A *(continued)*

3. Concrete Perimeter Beam (10″ × 24″)	250 plf
4. Gabled Roof (¾″ plywood, 2″ × 6″ rafters 2 ft OC, 225 lb compression)	5–10 psf
Perimeter Load:	
25′ × 40′ = 1380 ft^2 roof = 9660 lb (7 psf); 9660 × 0.75 = 7245 lb ÷ 130 ft	55 plf
Interior Load:	
9660 × 0.25 = 2145 lb ÷ 90 linear ft X-beams	24 plf
5. Walls:	
2″ × 4″ studs/plates, 16′ OC, 3/8″ sheet rock, two sides	6 psf
8′ wall × 4″ thick = 48 × 1/3 or 16	16 plf
2″ × 4″ stud wall, 3/8″ sheet rock one side	4 psf
8′ wall × 4″ thick = 32 × 1/3 or 10.67	11 plf
Brick Veneer	40 psf
8′ wall × 4″ thick = 320 × 1/3 or 107	107 plf
6. Ceilings	
2″ × 6″ joists, 16″ OC, 3/8″ sheet rock, one side	6 psf
12 ft × 13 ft room = 156 ft^2, 50 linear ft wall support	
Wall Load 936 lb ÷ 50 ft	19 plf
Load on Perimeter	5 plf
Single Story Loads	
Concrete Perimeter Beam	
(250 + 11 + 107 + 55 + 5)	428 plf
Concrete Interior Cross Beams	
(12′ OC)(165 + 16 + 19+ 24)	224 plf
Concrete Slab	50 psf
Live Load	40 psf
Second Story—Structural Load on Foundation	
Interior Cross Beams (224 + 16 + 19)	259 plf
Perimeter Beam (428 + 107 + 5 + 16)	556 plf
Live Load, Both Floors	70 psf

Source: D. Ramsey, *Foundation and Floor Framing,* McGraw-Hill, New York, 1995.