SECTION 7B
FOUNDATION REPAIR PROCEDURES

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This section deals with the many and various aspects of foundation repair. The discussion is divided into areas of principal concern.

7B.1 Addresses supporting interior floors for pier-and-beam type foundations.

7B.2 Covers mudjacking required for slab foundations.

7B.3 Provides a cursory discussion of grouting. This material is relegated to applications involving lightly loaded foundations and soil problems generally less than 30 ft (9 m) in depth.

7B.4 Discusses the various options for underpinning, with emphasis on the relative effectiveness of each method.

7B.5 Addresses basements and foundation waLl repairs.

7B.6 Provides a cursory discussion of soil stabilization as a remedial or preventive tool for problems dealing with lightly loaded foundations.

7B.7 Presents interesting case histories.

7B.8 Discusses the perimeter aspects for estimating.

7B.1 SUPPORTING INTERIOR FLOORS (PIER-AND-BEAM FOUNDATIONS)

7B.1.1 Introduction

Foundation repairs are generally categorized by cause—settlement or upheaval. Pier-and-beam foundations, generally, are more susceptible to settlement problems. Of the two foundation problems, upheaval is by far the most prevalent and more costly to repair.[15–17,26*]

Regardless of the cause of failure, the approaches to normal repair are quite similar. Whether a perimeter has settled or the interior heaved, the normal repair procedure is to underpin (raise) the perimeter (although rarely, the perimeter can be lowered[15–17]). Interior floors are then "leveled" by shimming the interior pier caps or, in the case of slabs, mudjacking. Underpinning will be discussed in Section 7B.4 and mudjacking in Section 7B.2.

7B.1.2 Shimming Existing Concrete Pier Caps

Leveling interior floors by shimming on existing concrete pier caps is the simplest of all foundation repairs. This is a problem characteristic of pier-and-beam foundations. Several times during the life of a residence, the need to level interior floors may arise. This can be considered as "routine mainte-

*References to this section are in Section 7E.

nance," much the same as repainting wood surfaces. Provided the existing piers are sound and properly located and assuming access, shimming requires nothing more than raising the girders to the desired elevation and placing wedges or shims on top of the pier caps to retain the position (Figure 7B1.1). Dimension hardwood or steel are used for major gaps. Cedar shingles are often used to fine grade.[15–17] Some people are confused regarding the advisability of using shingles. The concern is not founded in fact. True the shingle will compress under the load of the structure. However, this compression will continue only to the point where differences in densities are met. This compression is taken into account by the contractor. Without the use of shingles, fine grading (less than ¼ to ⅜ in (0.6 to 0.95 cm) is not as practical.[16] Also bear in mind that shingles have been used in new construction for over 50 years to level plates, frames, and girders. The use of thin steel shims can be a viable option.

7B.1.3 Shimming on Supplemental Pier Caps

When existing floor supports are deficient for one reason or another, the need for new pier caps arises. The ideal solution would be the installation of "new" drilled piers, as shown in Figure 7B.4.1. However, this approach is prohibitively expensive for existing structures because of restricted ac-

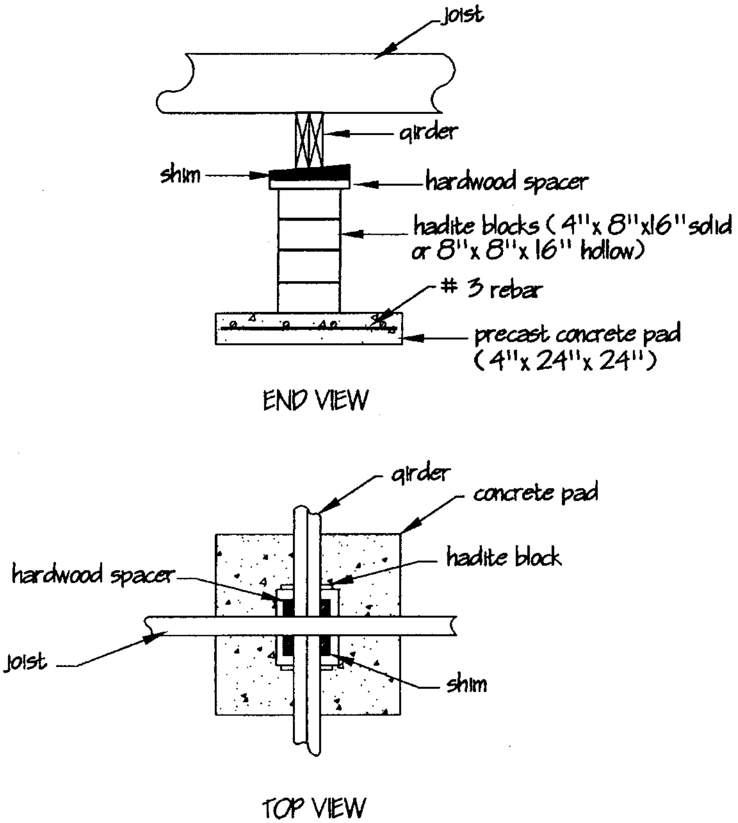


FIGURE 7B.1.1. Supplemental interior pier cap (typical).

cess. The flooring and perhaps partition walls and joist would have to be removed to provide access. Thus, supplementary supports will normally consist of precast concrete pads (leveled into the soil surface) with masonry blocks (usually Hadite) serving as a pier cap. Refer to Figure 7B.1.1.

The support depicted in Figure 7B.1.1 involves a concrete base pad, a concrete pier cap, suitable hardwood spacers, and a tapered shim for final adjustment. The base pad can be either poured in place or precast. The choice and size depends largely upon the anticipated load and accessibility. For *single-story frame* construction, the pad is normally precast, at least 18 × 18 × 4 in thick (46 × 46 × 10 cm), with or without steel reinforcing.

For *single-story and normal two-story brick-construction* load conditions, the pad should be steel-reinforced and at least 24 × 24 × 4 in thick (61 × 61 × 10 cm). For unusually heavy load areas, e.g., a multiple-story stairwell, the pad should be larger, thicker, and reinforced with more steel. In the latter case, the pad is normally poured in place. (The added weight creates severe handling problems for the precast pads). In any event, the pad is leveled on or into the soil surface to produce a solid bearing. Conditions rarely warrant any attempt to place the pads materially below grade. Often these may be 3′ × 3′ × 6″ thick (0.9 × 0.9 × 0.15 m) and reinforced with two mats of #3 rebar. Once the pad is prepared, the pier cap can be poured in place or precast; in most cases, the limited work space favors the precast cap. Choices for the precast design include concrete cylinders, Hadite blocks (lightweight concrete), or other square masonry blocks. (Ideally, the head of the pier cap should be at least as wide as the girder to be supported.) Either material is acceptable for selected loads. The Hadite blocks, for example, may occasionally be inadequate for multiple-story concentrated loads unless the voids within the blocks are filled with concrete or solid blocks are utilized. [As an aside, the crushing strength of a hollow 8 × 16 in (20 × 40 cm) Hadite block is about 110,000 lb. (49,900 kg).] The need for shimming to correct interior settlement is often recurrent, since the true problem (unstable bearing soil) has not been addressed. However, reshimming is relatively inexpensive, and the rate of settlement decreases with time because, at least in part, the bearing soil beneath the pier is being continually compacted.

7B.1.4 Replacing Substandard Floor Supports (Pier Caps)

When a structure is supported on wood "piers" or stiff-legs, it is rarely justified to replace only a portion of the wood supports with the superior masonry or concrete design. Normally, such a practice would 1) represent little, if any, long-range benefit, 2) represent a waste of money, and 3) possibly prevent a future HUD insured loan.

Although most foundation repair concerns are directed toward foundations of concrete design, there exists the often neglected issue of frame structures on wood foundations. The discussion about to unfold will address several aspects of foundation repair inherent to the frame design. It will become apparent that the foundation repair may involve: 1) leveling of floor systems basically as is, 2) leveling the floors and creating *minimal* clearance between the wood substructure and ground, or 3) providing *adequate crawl space* with concurrent leveling.

7B.1.5 Frame Foundations with Limited Access

Wood members near to (or in contact with) the ground are obviously influenced more by moisture than those farther removed and protected by air circulation. Contact with moisture encourages the wood framing to warp and rot.

Differential movement over a prolonged period of time exacerbates the same warp. In addition, the limited access to interior floor joists and girders complicates remedial activities. In the end, each condition represents serious deterrents to foundation repair. In other words:

1. Limited or nonexistent crawl space must be addressed. Access is required not only for wood replacement and foundation leveling but also for access to utilities and inspections. In some instances, crawl space of 18 in (0.45 m) is specified. The latter represents yet another problem, as

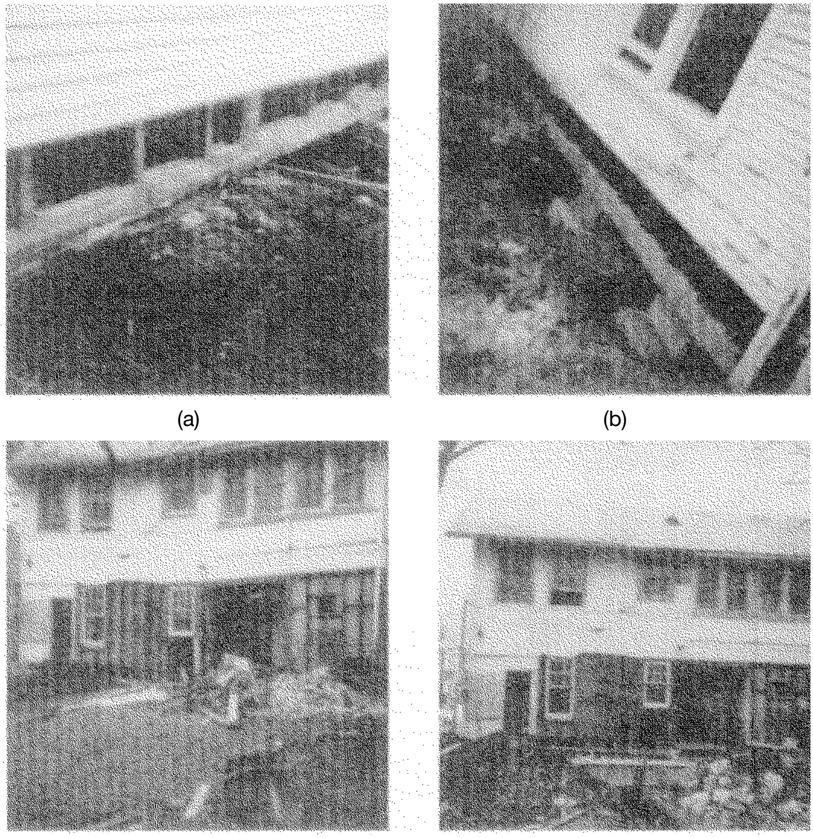
will be discussed in following paragraphs. For limited access, moderate tunneling can generally provide the access required for simple jobs.

2. Preconditioned warp in the wood substructure must be understood and compromised. Most often "leveling" is neither practical nor possible.
3. Relative costs for various alternatives to provide access are sometimes prohibitive. Extreme causes could involve removing the floors and in some instances even wall partitions.

Figure 7B.1.2 is a photographic review of a frame foundation before, during, and after repair.

7B.1.5.1 Providing Access

This situation offers several alternatives, none of which are easy or inexpensive. For example, the interior floors and perhaps some partition walls can be removed as required to provide access to the floor substructure. Concurrently, the skirting can be removed to provide access to the perimeter sill plates. This done, the structure can be "leveled" (or raised) to create a desired crawl space. Tunnel-



(a) (b) (c) (d)

FIGURE 7B.1.2. Repair to frame foundation with limited crawl space. (a) Frame foundation on soil is badly deteriorated. (b) New wood beam is in place and supported above grade on masonry pads and piers. (c) Prior to repair: note very extensive sag at doorway and roof eaves. (d) After repair: doorway and roof lines are straight.

ing, although expensive in itself, can be an option. Once the access is provided, masonry pier caps can be installed. Refer to Figure 7B.1.1 for a typical pier cap design. If the foundation is to be raised to provide the 18 in (0.45 m) crawl space, concern must be given in advance to utility connections in order to circumvent unnecessary damage to water, sewer, or electrical connections. Hand excavation is expensive. The excavation to provide 18 in (0.45 m) crawl space for a 1000 ft^2 (93 m^2) house would remove 55 yd^3 (42 m^3). Some contractors figure that a single laborer can tunnel 2.0 yd^3 (1.5 m^3) in an 8 hour day. At an hourly rate of \$6.50 hour, the example would cost \$5700.00. Care must be taken in the use of the word "level." This term is not applicable to foundation repair procedures in general and even less so when old frame substructures exhibit warp.

Alternative choices include the use of steel beams to raise and support the structure independent of the foundations. (Placement of the beams are generally aligned to utilize any existing girder/perimeter support system.) As a rule, house movers are better equipped to handle this operation. Once the structure is elevated, two options become available. First, if lateral space permits, the structure can be moved aside to provide access to drill piers and set forms for installation of new concrete piers, pier caps, and perimeter beam. Second, if lateral space is not available, the fact that the structure is elevated will permit some work to be done. Precast masonry piers and pads can be located to support the lightly loaded interior floors. Conventional spread footings, concrete piers with haunches and pier caps, or continuous concrete perimeter beam can be poured in place to support the exterior. Once the foundation has been constructed, the house can be reset.

In summary, for most cases the most favorable option is to remove sections of floor and excavate soil to provide the desired access and minimal clearance between the wood substructure and ground. This approach is often the least expensive. The expensive solutions normally involve those instances where excessive excavation is required for the creation of a complete crawl space or the structure is literally moved to permit the installation of a foundation. Many problems are resolved once the foundation is raised by whatever means and supported by a masonry/concrete foundation. Access, ventilation, and insulation from the ground are handily resolved. The final goal is to "level" the floors. This ambition is often met with compromise, since warped wood subjected to no appreciable load will not likely relax or straighten. A high spot left unsupported might, over time, settle somewhat to project a degree of "leveling"; however, this event is by no means predictable. Generally, the pier caps are shimmed to provide a "tight" floor. The bottom line is that the end results are clearly a degree of compromise.

7B.1.5.2 Foundations with Adequate Access

This situation provides a much better, less expensive set of approaches, as discussed in following paragraphs.

7B.1.5.3 All-Frame Foundations

If all foundation support members are frame, two options exist: (1) the floor system (including perimeter) can be reshimmed on the existing wood piers or (2) the wood piers can be completely replaced by masonry pier caps on concrete pads. Refer to Figure 7B.1.1.

7B.1.6. Concrete Foundations

Floor systems with a girder-wood substructure, supported in turn by a concrete pier-and-beam foundation, represents a fairly easy challenge (Figure 7B1.3). The floors are releveled by raising the girders. The girders are in turn held in position by reshimming on the existing pier cap. Refer to Figure 7B.1.1.

7B.1.6.1 Concrete Perimeter with Frame Interior Piers

For foundations with concrete perimeter beams and wood interior piers, the interior wood piers can be either shimmed as is or replaced, as before, with masonry pier caps and pads. Problems with the perimeter beam are classically addressed by conventional underpinning (refer to Section 7B.4).

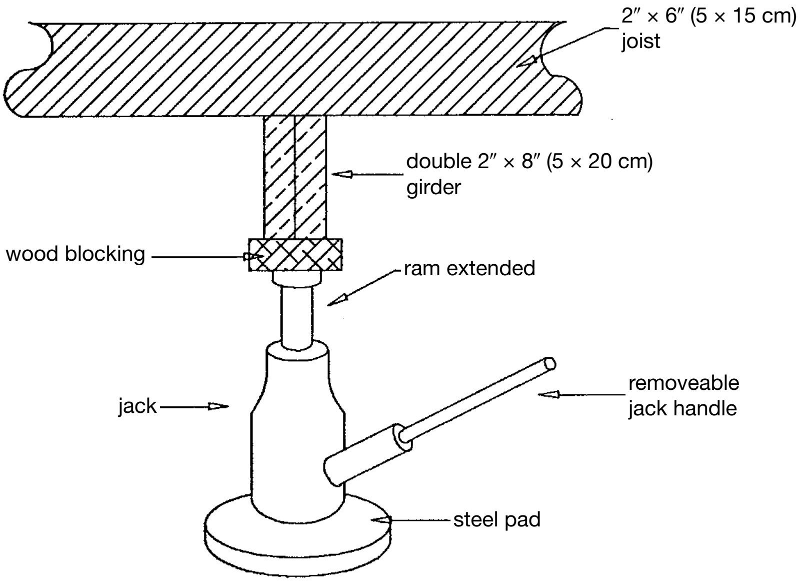


FIGURE 7B.1.3. Jack in place to raise girder.

7B.1.7 Equipment Necessary and Raising Procedure

The installation procedures are much the same, whether the project is to shim the existing foundation or install supplementary pier caps. In order to "level the playing field," the following will assume ready access.

7B.1.7.1 Selection of Jacks

The first equipment to select is the jack needed to provide the force necessary to lift the structural weight. Frequently, Norton or Simplex journal or hydraulic jacks are utilized for this operation. These jacks are about 13 in (33 cm) in length, have a 5 in (13 cm) extension, and are rated at 25 ton (22,700 kg) capacity. Frequently, the choice is an aluminum body hydraulic jack that weighs and costs about one half as much. A normal "leveling" operation could require four to twelve jacks.

7B.1.7.2 Positioning the Jacks

The jacks are strategically placed beneath the wood members to be raised. Refer to Figure 7B.1.3. If the soil is soft, the jacks should be placed on a steel plate, wood blocks, or both to prevent the jacks from sinking into the dirt. A steel plate could be a circular piece, ⅜″ (9.5 cm) thick, and 20″ (50 cm) in diameter with a handle welded to one or two sides. The blocks are normally pieces of hard wood 2″ to 4″ (5 to 10 cm) thick and of various widths and lengths. The same wood is also used to block on top of the jack head to both preserve the extension and spread the lifting pressure. Figure 7B.1.4 illustrates jack placement used to install a single girder. Figure 7B.1.5 addresses the problem of installing multiple girders. In order to raise the floor position, a watch person is stationed inside the area to be raised; upon his command, the four jacks at position B raise the floor about 1 in (2.54 cm). Next the jacks at A and C are raised about ⅜ ″ to ½″ (0.9 to 1.25 cm). This process is repeated

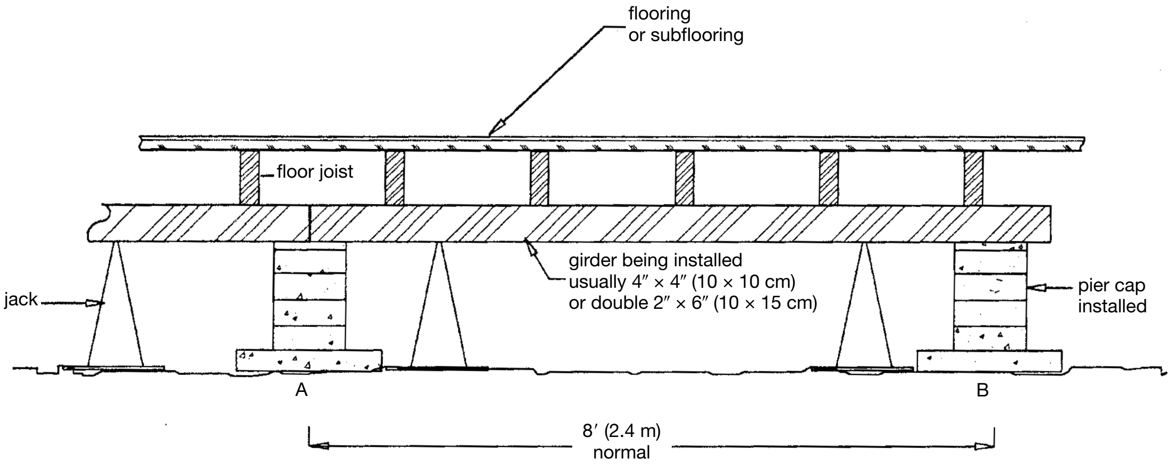


FIGURE 7B.1.4. Installing a wood beam to act as a girder to support floor joists. Note splice at pier cap A. Jacks are removed when pier caps have been installed

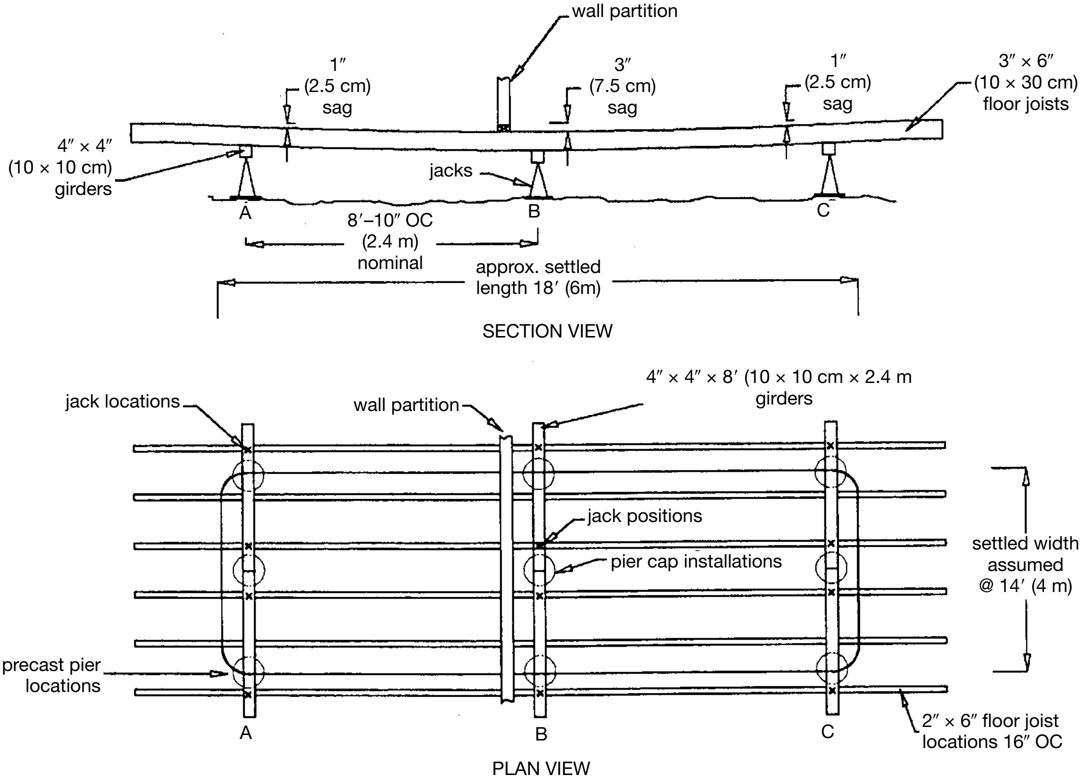


FIGURE 7B.1.5. Positioning supports for leveling floor sag. Refer to Figure 7B.1.4 for detail at splice.

until the maximum amount of leveling is achieved. Then precast pier caps on pads are installed to sustain the floors.

If the desired raise exceeds the extension capacity of the jack, it becomes necessary to 1) temporarily block the girder, 2) compress the ram, 3) add blocks and reposition jack, and 4) repeat the original process.

7B.1.7.3 Leveling the Foundation (Floor Joists)

When supplementary floor support is necessary, the process usually involves the addition of girders supported on masonry pier caps, as described above. Figures 7B.1.4 and 7B.1.5 depict such operations. Occasionly, the need to supplement floor joists is also involved. On even rarer occasions, the responsible approach might require the installation of steel "I" beams or channel iron. Refer to Figure 7B.1.6.

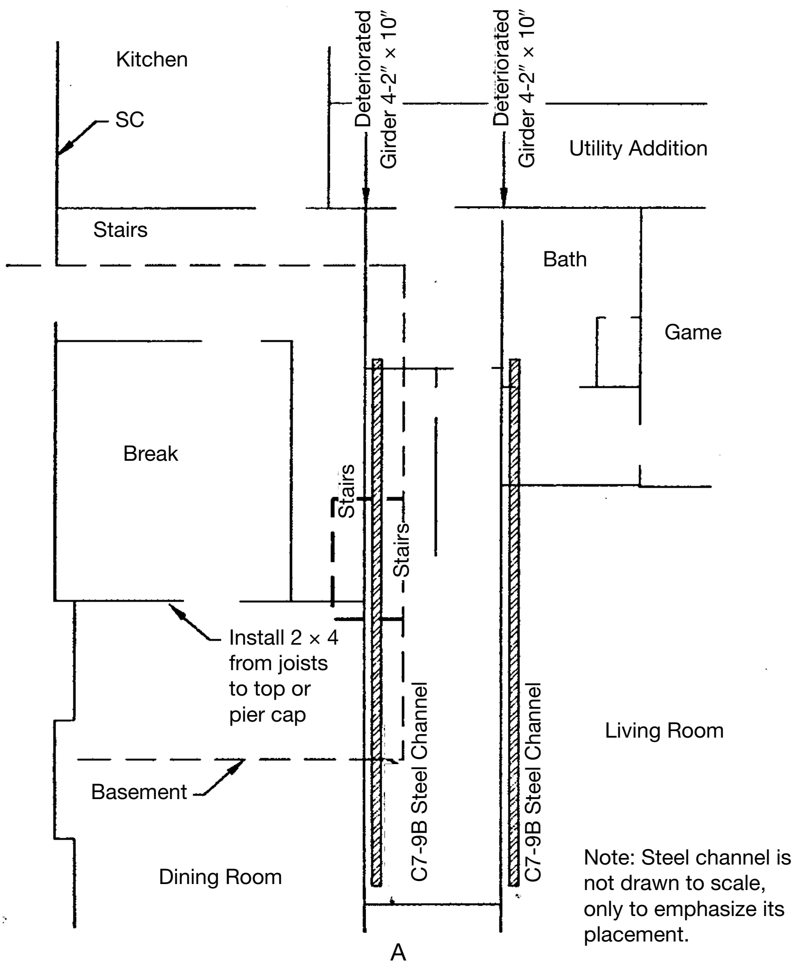


FIGURE 7B.1.6. (A) Floor plan showing placement of channel iron. (Figure continues)

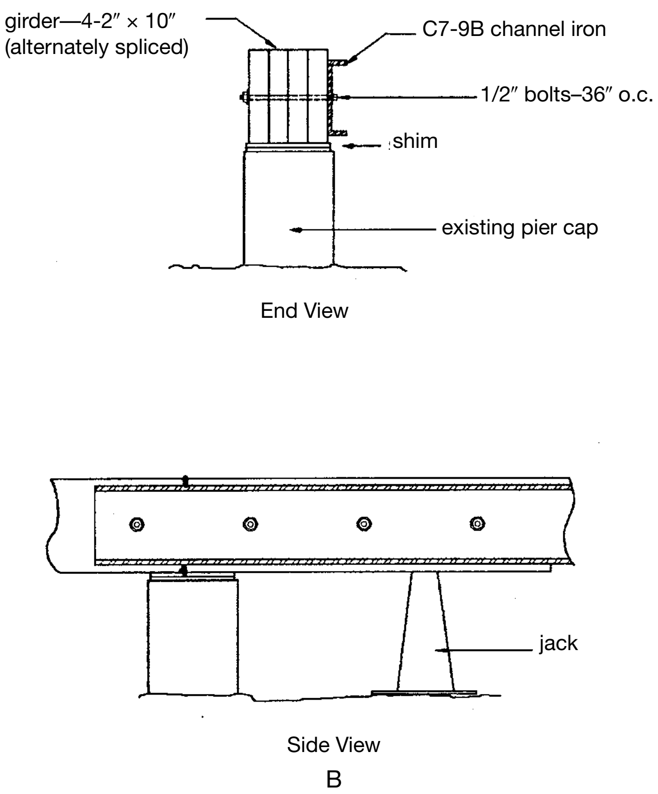


FIGURE 7B.1.6. (*continued*) (B) Installation of channel iron to reinforce wood girders.

7B.1.7.3.1. Relative costs for conventional leveling procedures can be summarized as follows:

Installation	Costs, $
Installation of girders	25/b. ft.*
Scab floor joists	25/b. ft.
Install masonry precast pier caps: assume maximum height of 18 in (45 cm):	
2 × 2 × 4 in (60 × 60 × 10 cm) pads	120 each
1½ ft × 1½ ft × 4 in pads (0.45 × 0.45 × 10 cm)	100 each
The cost for cast-in-place pads or pier caps depends to a large extent on access: assuming at least an 18 in (0.45 m) crawl space:	
30 × 30 × 9 in (75 × 75 × 24 cm) pad	190 each
12 in (30 cm) diameter pier cap	48 each

*b. ft. = board feet. One board foot is represented by a 1″ × 12″ board one foot in length.

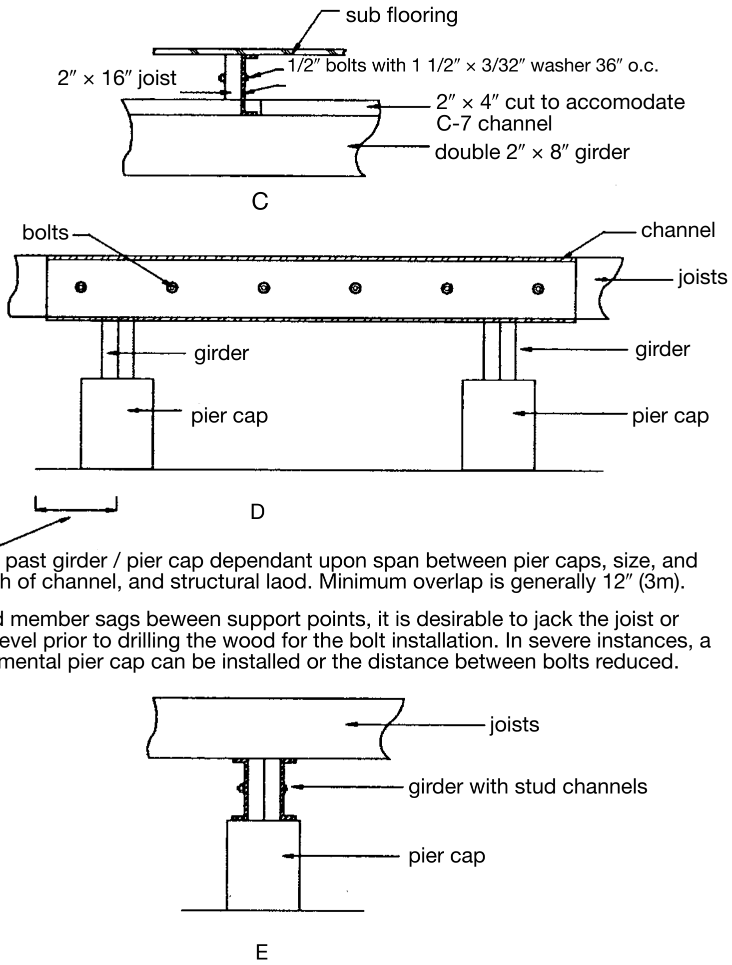


FIGURE 7B.1.6. (*continued*) (C, D, & E) Channel iron used to stiffen joists or girders.

The cost for an example installation of channel beams to reinforce joists or girders is much higher than the installation of supplemental wood. This cost is approximated as follows (refer to Figure 7B.1.6C and Table 7B.1.1):

C-4 (7.25 lb)	$ 34/linear foot
C-5 (6.7 lb)	$ 40/linear foot
C-7 (9.8 lb)	$ 60/linear foot
C-8 (11.5 lb)	$ 53/linear foot

TABLE 7B.1.1 Weights and Dimensions of Structural Steel Channels

Depth of channels, inches	Weight, lb/ft	Thickness of web, inches	Width of flange, inches
3	6.00	0.362	1.602
	5.00	0.264	1.504
	4.10	0.170	4.410
4	7.25	0.325	1.725
	5.40	0.180	1.580
5	9.00	0.330	1.890
	6.70	0.190	1.750
6	13.00	0.440	2.160
	10.50	0.318	2.038
	8.20	0.220	1.920
7	14.75	0.423	2.303
	12.25	0.318	2.198
	11.50	0.220	2.260
	9.8	0.210	2.09
8	18.75	0.437	2.260
	13.75	0.303	2.343
	11.5	0.220	2.522
9	20.00	0.452	2.652
	15.00	0.288	2.488
	13.40	0.230	2.430
	30.00	0.676	3.036
10	25.00	0.529	2.889
	20.00	0.382	2.742
	15.30	0.240	2.600
12	30.00	0.513	3.173
	25.00	0.390	3.050
	20.70	0.280	2.940
15	50.00	0.720	3.720
	40.00	0.524	3.524
	33.90	0.400	3.400

Source: *The Building Estimators Reference Book*, F. R. Walker Co., 1970. Prices are installed and bolted through the wood joists on 12″ (0.3 m) to 30″ (0.75 m) centers. Channels are predrilled by supplier. Prices assume reasonable access and channel total weights less than about 150 lb (68.2 kg). The unit prices will fluctuate slightly depending upon spacing and diameter of bolts, access, current steel and labor prices, weight of channel, etc. The cited prices include ½″ (1.25 cm) bolts on 24 in (0.6 m) centers. The material costs for the C7 × 9.8 channel iron drilled, cut, and delivered was $6.50 U.S. per linear ft (1999 U.S. dollars)

Table 7B.1.2 provides weight and dimension values for “I” beams. The foregoing was based on the following assumptions:

1. All concrete to be reinforced with #3 rebar (0.9 cm).
1. All prices quoted are based on unskilled labor at $7.00/h and concrete at $60/yd^3.
3. No extraneous restrictions or interferences.

Note that the use of structural steel to reinforce or supplement wood substructure members (joists or girders) to support interior floors on residential pier and beam foundations, is broadly considered to be a grossly expensive overkill. The alternative use of wood should be evaluated.

TABLE 7B.1.2 Weights and Dimensions of Structural Steel "I" Beams

Section	Wt, lb/ft	CSA, in^2	Section width, inches	Flange		Web thickness inches
				Width, inches	Thicknes, inches	
W-6 x	16	4.74	6.28	4.03	0.405	0.260
	12	3.55	6.03	4.0	0.280	0.230
	9	2.68	5.9	3.94	0.215	0.170
W-8 x	15	4.44	8.11	4.015	0.315	0.245
	13	3.84	7.99	4.00	0.255	0.230
	10	2.96	7.89	3.94	0.205	0.170
W-10 x	19	5.62	10.24	4.02	0.395	0.250
	17	4.99	10.11	4.01	0.330	0.240
	15	4.41	9.99	4.00	0.270	0.230
	12	3.54	9.87	3.96	0.210	0.190
W-12 x	22	6.48	12.31	4.03	0.425	0.260
	19	5.57	12.16	4.005	0.350	0.235
	16	4.71	11.99	3.99	0.265	0.220
	14	4.16	11.91	3.97	0.225	0.200

Source: Armco Steel Corporation, Houston, Texas.

Tables 7B.1.7.3 and 7B.1.7.4 provide a basis for strength (design) comparisons. Cost comparisons are provided in Section 7B.1.7.3.

7B.1.8 Sizing Beams (Girders) for Support of Interior Floors

Upon occasion, the existing wood beams are incapable of providing adequate support for the floor joists/interior floors. This may be caused by damage to the original beams (rot, splitting, torsional rotation, etc.), increase in structural load (live loads or dead loads beyond anticipation, additions, etc.), or faulty design. When any of these occur, the existing wood beam members must be reinforced. Classically, this is accomplished by removing and replacing the defective wood, scabbing existing wood with new wood (of identical size), or, occasionally, installing. (In heavy construction, "I" beams and sometimes even "H" beams are used.) The following discussions will help provide design characteristics suitable to allow "best use" evaluations of the options.

7B.1.8.1 Wood Beam Design Features

Prior sections have touched on accepted repair procedures common to the industry. However, at best, only limited information was provided as a basis or guide for the actual remedy necessary. The following information is offered as a *guide* to assist with the repair of residential foundations. This information is *not* intended to be used for structural design purposes.

Tables 7B.1.7.3a and b provide lumber design data and actual loads for determining the required support (beams) for interior floors. The example calculations present a mathematical analysis for a 2″ × 8″ (10 × 40 cm) member supported by at least three continuous piers (pier caps) at the indicated spacing. (Note that the 2″ × 8″ board is actually 1½″ × 7¼″). All calculations are based on the following:

- Dead load + live load = 60 lb/ft^2
- Beams are spaced 12 ft OC
- Beams are supported on three or more continuous piers (pier caps)

TABLE 7B.1.7.3a Design Factor for Selected Wood [S4S Dry Lumber, (< 19%), Southern Pine, #1 Grade]

	Dimension, inches	CSA, in^2	I_x, in^4	S_x, in^3	F_B, psi	F_V, psi	E, psi
2″ × 4″	1½ × 3½	5.25	5.36	3.06	1850	180	1.7×10^6
2″ × 6″	1½ × 5½	8.25	20.80	7.56	1650		
2″ × 8″	1½ × 7¼	10.88	47.63	13.14	1500		
2″ × 10″	1½ × 9¼	13.88	98.93	21.39	1300		

- For Grade 2 Lumber, multiply F_B by 0.80
- $M_B = wl^2/10 \times$ ft/12 in $= wl^2/120$ in-lb, $Fb = M/S_x$, psi

(Note: If beams are supported only at two ends, the appropriate moment equation is: $M_B = wl^2/8 \times 12$. Following bending moment calculations for continuous beams supported by three or more pier caps can be corrected for a beam supported at two ends by multiplying the following calculated bending moments by a factor of 1.25.)

Beam stability is approximated by the following depth-to-width ratio, d/b:

1. $d/b = 2{:}1$ or less, no lateral support required
2. $d/b = 3{:}1$ or $4{:}1$, the end must be held in position to prevent lateral rotation
3. d/b 5:1, one edge of the beam must be held in line for its entire length

From Table 7B.1.7.3c, the allowable working load (F_B) compared to the actual load (f_B) suggests that for an 8 ft span, the beam should consist of three 2″ × 8″ members. (These would be nailed together with joints staggered.) The defection (Δ), based on an acceptable defection of 1/360, would require two 2″ × 8″ boards. The shear (V) requires that the beam be composed of three 2″ × 8″ boards. To safely accommodate the design load both the allowable working load and the shear dictate the use of three 2″ × 8″ boards. In analyzing the design, the *weakest* factor dictates. Figure 7B.1.7 depicts the load distribution on a typical beam. For this example, $w = 60$ lb/ft^2 × 12 ft or 720 lb/ft.

Example Calculations (Wood Beams)

$M = wl^2/10 \times$ ft/12 in $= wl^2/120$ in lb
$f_B = M/S$, psi

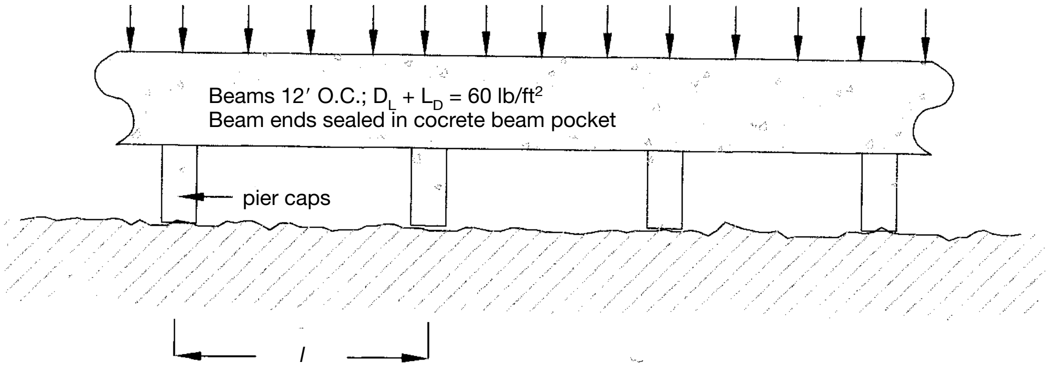


FIGURE 7B.1.7. Beam load distribution.

$V = 5wl/96$, lb
$fv = 1.5V/A$, psi
$S = \text{bd}^2/6$, in^4
$\Delta = wl^4/1743\ EI$, in; $I = \text{bd}^3/12$, in^4

2″ × 8″ Wood
8 ft. span (96″) (96 in)^4 = 84.93 × 10^6 in^4

Bending Moments/Working Stress, f_B
$M = 720\ (96)^2/120 = 55{,}296$ in-lb
From Table 7B.1.7.3a: $S = 13.14$ and $F_B = 1500$ psi
$f_B = 55{,}296/13.14 = 4208$ psi, ∴ fails
Number required = 4208/1500 = 2.8, needs 3 2″ × 8″

Stability
$d/b - 7.25/1.5 - 4.8{:}1$, needs at least one end held in place, which is satisfied

Deflection (Δ)
$\Delta = wl^4/1743\ E \times I$ $\quad I = bd^3/12 = 47.63$ in and $E = 1.7 \times 10^6$ (from table 7B.1.7.3a)
$= 720\ (96)^4/1743(1.7 \times 10^6)(47.63) = 0.433$ in, ∴ fails
Allowable deflection: * l/360 = 0.267 in
l/270 = 0.355 in
Number 2″ × 8″ × 8′ required for Δ: 0.433/0.267 = 1.62 or 2

Shear (V)
$V = 5wl/96 = (5)(720)96/96 = 3600$ lb
$f_V = 3/2\ V/A = (3/2)(3600)/10.88 = 496$ lb/in^2, fails
$F_V = 180$ lb/in^2, hence, $F_V < f_V$, therefore failure in shear
Number required: 496/180 = 2.75 or 3

2″ × 8″ Wood Beam (*continued*)
10 ft span = 120″ (120)^4 = 207.36 × 10^6 in^4

Bending Moment/Working Stress (f_B)
$M = 720\ (120)^2/120 = 86{,}400$ in-lb
$f_B = 86{,}400$ in-lb/13.15 = 6575 psi
Number required: 6575/1500 = 4.3 or 5

Stability
$d/b = 7.25/1.5 = 4.8{:}1$, needs at least one end held in place or one edge nailed in place

Deflection (Δ)
$\Delta = 720\ (207 \times 10^6)/EI(1743)$
$= 14.9 \times 10^{10} / (1743)\ (47.63)\ (1.7 \times 10^6)$
$= 14.9 \times 10^{10} / 14.2 \times 10^{10} = 1.05$ in
1.05 > 0.267 ∴ fails

Shear (V)
$V = 5\ wl/96 = (5)\ (720)\ (120)/96 = 432{,}000/96$
= 4,500 lb
$f_V = 3/2\ V/A = 3/2\ (4{,}500)/10.88$
= 620.4 lb/in^2
f_V (620.4) > F_b (180) ∴ fails
Number required for load = 620/180 = 3.4 or 4

*Note that the conservative Δ will be used throughout (0.267 in).

In many cases, the floor joists are 2″ × 6″

2″ × 6″ wood beam
8 ft. span (96″) (96 in^4) = 84.93 × 10^6 in^4

Bending moment/working stress
M = 55296 in-lb, from prior calculations
f_b = 55296/7.56 = 7314 psi
Again, since F_b = 1500 psi, ∴ fails
Then number 2″ × 6″ required for load = 7314/1500 = 4.8 or 5

Stability
d/b = 5.5/1.5 = 3.7 or 4
At least one end must be held in place or edge nailed in place

Note: With floor joists, stability is generally not an issue since the subflooring is nailed into joist edge.

Deflection (Δ)
$\Delta = wl^4/1743EI$
Δ = (720) 84.93 × 10^6/1743 (1.76 × 10^6) 20.8
= 61146.6 × 10^6 / 63807.7 × 10^6
Δ = 0.95″, which exceeds allowed valve of 0.267
Number required for defection 0.95/0.267 = 3.6 or 4

Shear (V)
$V = 5wl/96$ = 5(720) (96)/96 = 3600 lb
f_v = (1.5) (3600)/8.25 = 655 lb/in^2, which exceeds F_b ∴ failure
Number 2″ × 6″ to resist shear: 655/180 = 3.6 or 4

Maximum allowable working loads
Calculate maximum allowable working load, $w_u = (F_B)(S_x)(120)/l^2(F_B \times S_X = M_u = w_u l^2/120)$

w_u = (1500)(13.14)120/9216 = 257 plf (8ft span)
w_u = (2.3652 × 10^6/5184 = 456 plf (6ft span)
w_u = 2.3652 × 10^6/14400 = 164 plf (10ft span)

These data are summarized in Table 7B.1.7.3b. Note that none of the beams can accommodate the design conditions and multiple timbers are required. Refer to Table 7B.1.7.3c.

Table 7B.1.7.3c also suggests that when the interior floors on a pier-and-beam foundation fail due to structural load, the repair contractor should check the support provided by the beams (girders). When the beams are on 12 ft (3.6 m) spacing and supported by pier on 8 ft centers, 3 2″ × 8″ boards (5 × 20 cm) (in undamaged condition) are required for safe support. If the beam consists of less than 3 2″ × 8″ (or if one or more of the members are damaged), the beam should be brought up to the designated standard. This would require also that: 1) the pier diameter supporting the beam is

TABLE 7B.1.7.3b Various Pier Spans*

	6 ft.		8 ft.		10 ft.	
	F_B, psi	w_u, plf	F_B, psi	w_u, plf	F_B, psi	w_u, plf
2″ × 6″	4,114	289	7,314	162	11,429	104
2″ × 8″	2,367	456	4,208	256	6,575	164
2″ × 10″	1,454	644	2,445	361	4,039	232

*Assumes: 60 psi total load ($LL + DL$); 12 ft beam (girder) spacing; S4S, Yellow Pine, Grade 1 < 19% moisture; w_u maximum allowable working load, plf.

TABLE 7B.1.7.3c Combination of Beams Required*

	6 ft span	8 ft span	10 ft span
2″ × 6″	3	5	7
2″ × 8″	2	3	5
2″ × 10″	2	2	4

*Assumes: 60#/ft (*DL* + *LL*); 12 ft beam spacing; Grade #1, Y.P., S4S, > 19% moisture; both ends of beam secured.

at least 4″ (10 cm) larger than the width of the beam (3 × 1.5″ = 4½″), 2) the beam is reasonably centered on the pier caps, and 3) the beam is stable (no torsional flex) on the pier caps. In the case of a 3 2″ × 8″ beam, the $d/b = 7.25/4.5$, which is less than 2:1. No lateral support is necessary.

7B.1.8.2 Channel Iron Application Analysis

Alternate beam materials could be considered. For example C-7 × 9.8 # channels could be considered when the desired 8″ (20 cm) wood cannot be installed. However, the channel irons should be bolted to *both* sides of the wood to avoid lateral torsional buckling. (This effect is much less a problem with beams [combination of several members] than it is with joists [single wood members]). When it can be safely avoided, the use of channel iron represents an unnecessary cost. The cost of channel is about three times more than the corresponding wood. However, in some situations, their use might be advantageous. For this possibility the following design charts, calculations, and comments should prove helpful.

(Note: The calculations used in the following paragraphs, are intended *solely* as a *tool* for comparative selections of materials to use for the structural support of wood floors. These calculations are grossly over simplified and should *not* be used for *design* purposes. Specifically, the use of dissimilar materials in combination as well as the non rectangular "c" sections introduce complicated mathematical concerns.)

Table 7B.1.4a gives the design factors for selected channels. Figure 7B.1.8 illustrates support of floor joists by the beams and pier caps. Following are several example calculations.

Example Calculations

5″ Channel
8 ft span (96″) $(96'')^4 = 84.93 \times 10^6$ in^4

Bending Moment/Working Stress, f_B
$M = 720(96^2)/120 = 55{,}296$ in-lb
$f_B = M/S = 55{,}296$ in-lb$/3.0$ in$^3 = 18{,}432$ psi
$F_B > f_B = (20{,}000$ psi $> 18{,}432$ psi$)$, ok

Stability
$d/b = 2.86$, at least one should be held to prevent rotation

TABLE 7B.1.4a Design Factors for Selected Channels*

Designation	Web × Flange, inches	CSA, in^2	I_x, in^4	S_x, in^3	F_B, psi	F_V, psi	E, psi
C-5 × 6.7	5 × 1.75	1.95	7.4	3.0	20,000	11,400	29×10^6
C-7 × 9.8	7 × 2.09	2.85	21.1	6.0		17,600	

*$M = wl^2/12 \times 10$; $S = bd^2/6$; $F_B = M/S$; $w = 60$ lb/ft^2 × 12 ft (beam spacing) = 720 plf.

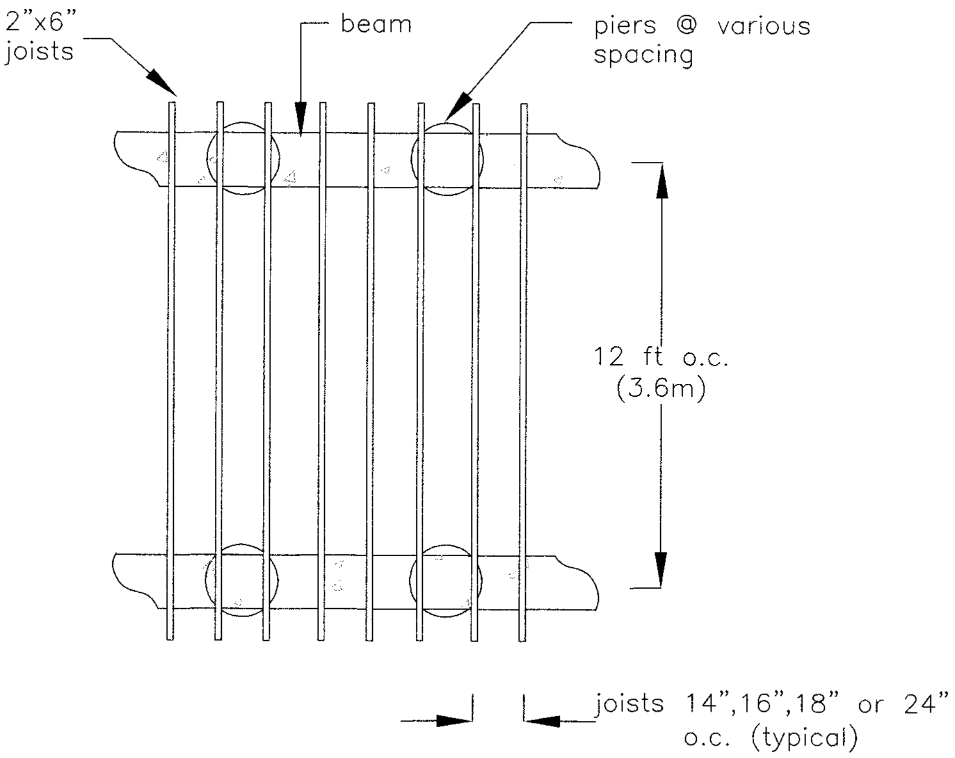


FIGURE 7B.1.8. Joists supported on beams.

Deflection (Δ)
$\Delta = wl^4/1743\ EI = (720)(84.93 \times 10^6)/(1743)(29 \times 10^6)(7.4)$ in $= 0.163''$
$0.163'' < 0.267''$ (allowable) $\therefore$ ok

Shear, V
$V = 5wl/96 = (5)(720)(96)/96 = 3600$ lb
$f_V = 3/2\ V/A = (1.5)(3600)/1.95 = 2796$ psi
$2796 < 11{,}400$ (allowable) $\therefore$ ok

Note: Based on a d/b of 2.86, one end of the channel iron should be supported to prevent rotation.

5″ Channel (*Continued*)
12 ft span (144″) $(144)^4 = 429.98 \times 10^6$ in^4

Bending Moment/Working Stress (f_B)
$M = wl^2/120 = (720)(144)^2/120 = 124{,}416$ in-lb
$f_B = M/S = 124416/3 = 41{,}472$
$41{,}472 > 20{,}000$ allowed $\therefore$ fails

Deflection (Δ) $(144)^4 = 430 \times 10^6$
$\Delta = wl^4/1743\ EI = (720)(430 \times 10^6/1743(29 \times 10^6)(7.4) = 0.828$ in
0.828 in > 0.266 in ∴ fails

Shear (V)
$V = 5wl/96 = (5)(720)(144)/96 = 5400$ lb
$f_V = 3/2\ V/A = (1.5)(5400)/1.95 = 4153$ psi
4153 < 11,400 allowed ∴ ok

7″ Channel
8 ft span 8 ft. × 12 in/ft = 96 in $(96\text{ in})^4 = 84.96 \times 10^6\text{ in}^4$

Bending Moment/Working Stress (f_B)
$M = 55296$ in-lb (from M calculations for 5″ Channel)
$f_B = 55{,}296/6.0 = 9216$ psi ∴ ok
$f_B < F_B$ ∴ ok

Stability
$d/b = 7/2.09$ (from Table 7B.1.4a) = 3.35, the channel must be held at one end

Deflection (Δ)
$\Delta = (720)(84.96 \times 10^6/(1743)(29 \times 10^6)(21.1) = 0.057''$

7″ Channel (*Continued*)

Shear V
$V = 3600$ lb (from V calculation for 5″ channel @ 8 ft)
$f_V = (1.5)(3600)/2.85 = 1894.74$ psi
$F_V = 17{,}600 > 1895$ ∴ ok

12 ft span (144″) $(144\text{ in})^4 = 429.98 \times 10^6\text{ in}^4$

Bending Moment/Working Stress (f_B)
M at 10 ft span was 86,400 in-lb, therefore M at 12 ft is $M = 86{,}400\ (144)^2/(120)^2$ or 124,416, in-lb
$f_B = 124{,}416/6 = 20{,}736$
$20{,}736 \gg 20{,}000$ ∴ Risky

Deflection (Δ)
$\Delta = 0.828$ in × 7.4/21.1 or 0.290 in
0.290 in ⋙ .267 in, allowed ∴ *risky*

Shear (V)
$V = 5400$ lb (from shear calculations re 5″ Channel at 12 ft)
$f_V = (1.5)5400/2.85 = 2842$ psi
2,842 < 17,600 allowed ∴ ok

Stability
$d/b = 7.2/2.09 = 3.34$ requires support at one end to prevent rotation

12 ft span: This happens to be the span frequently used with 2″ × 6″ *joists* considered in this study. (The 2″ × 6″ joist accomodates the C-5 × 6.7 channel).

(NOTE: The bending moment/applied stress for 2″ × 6″ joist on 18″ centers supported by beams 12 ft apart and assuming the 60 psf load would be $w = (60\text{ psf})(1.5\text{ ft}) = 90$ plf)

Bending Moment/Working Stress (f_B)
$M = wl^2/120 = (90)(144)/120 = 15{,}552$ lb
$f_B = M/S_X = 15552/7.56 = 2057$ psi (S_X from Table 7B.1.4b)
2057 > 1650 allowed ∴ fail

Allowed moment

$F_B = 1650$; $M_u = 1650 \times 7.56 = 12{,}474$ in-lb

then allowed load

$w_U = (12474)(120)/(144)^2 = 72$ plf

Allowed joist spacing then becomes

$72/60 = 1.2$ ft or 14 in

$d/b = 5.5/1.5 = 3.66$ Joist must be held at ends to prevent rotation or the joists must be nailed into the subflooring. Fortunately, the latter is normal for residential construction. When the joist fails in service, the remedial choices are to either scab the joist with another 2″ × 6″ or install a C-5 × 6.7 channel bolted through the failed joist. Actually, as stated earlier, the safest use of the channel would be to bolt one to either side of the joist. This prevents the threat of lateral torsional buckling.

Table 7B.1.4b presents a summary of the working stress calculations. Neither channel section would safely span the 12 ft distance typically required for a floor joist. Although the given channels are theoretically capable of sometimes supporting structural loads under the given condition, in actual use a single channel would not function as a beam even if the design cracks suggest otherwise. In practice, it would be necessary to bolt two channels back to back, possibly with a wood member of suitable size "sandwiched" between. Rather then using 3 2″ × 8″ (Table 7B.1.3c) at a cost of perhaps $48 per linear ft, the channel iron beam would cost about $75 plf.

In remedial situations, the support beam may be deficient due to perhaps one of the 2″ × 8″ boards being rotted or splint. In this case, the defective wood needs to be replaced. The economical approach would be to remove and replace the defective member with new lumber. Alternatively, the beam could be scabbed with a C-7 × 9.8 channel.

7B.1.8.2.1 Selecting the Proper Channel for New Installation. For new installations requiring channel irons, the proper selection can be handily facilitated through the use of Table 7.B.1.5. First select one of the example calculated in Section 7.B.1.8.2. For example the bending moment for a 12 ft span ($w = 720$ plf) is 124,416 in-lb. Given that $F_b = 20{,}000$ psi:

$$\text{Section Modulus } (S) = 124{,}416/20{,}000 = 6.22$$

$$S \times L = 12 \times 6.22 = 74.6$$

Consult Table 7B1.5 and select the first value greater than 74.6, which is 76. Now select the first channel iron with a modulus (s) greater than 6.22. The economical channel would be C-7 × 9.8. Check the deflection and stability of the channel iron to confirm it application.. Obviously, the same

TABLE 7B.1.4b Summary of Working Stress ($k = 1000$ lb)

Pier spacing	8 ft			10 ft			12 ft		
Channel	F_B, ksi	in	F_V, ksi	F_B, ksi	in	F_V, ksi	F_B, ksi	in	F_V, ksi
C-5 × 6.7	18.43	0.163″	2.8	28.8*	0.398*	3.46	41.42*	0.829*	4.15
C-7 × 9.8	9.216	0.057	1.9	14.4	0.14	2.37	20.74*	0.29*	2.84

*Exceeds design values.

Note: The analysis is focused on a single channel iron, which, of course, is impractical in the real world. However, the working stress values serve to provide a basis for both design concern as well as remedial requirement. In virtually all instances, the coupling of two channel irons would meet the design criteria. Pay attention to earlier statements regarding lateral torsional buckling.

TABLE 7B.1.5 Structural Steel

Table for economical selection of steel beams with unbraced flanges*—AISC 1947
Example: Given M = moment = 89 ft kip; L_u = unsupported length = 14 ft; steel working stress = 20,000 p.s.i.

To find economical wide-flange beams

Solution: S = section modulus = 89,000 × 12/20,000 = 53.4

$S \times L_u = 53.4 \times 14 = 747$

Enter table at first figure of SL_u greater than 747, which is 759. Select first beam with section modulus greater than required, 53.4. Beam is 16 WF 45. Check for deflection.

SL.	S	Section	L.	SL.	S	Section	L.	SL.	S	Section	L.	SL.	S	Section	L.
WIDE-FLANGE BEAMS				3380	242.8	27 WF 94	13.9	74	13.8	10 LB 15	5.4	435	37.8	12 I 35.0	11.5
142	14.1	8 WF 17	10.1	3420	156.1	19 WF 85	21.9	79	17.5	12 LB 16½	4.5	645	44.8	12 I 40.8	14.4
208	17.0	8 WF 20	12.2	3600	220.9	24 WF 94	16.3	91	11.8	8 LB 15	7.7	671	58.9	15 I 42.9	11.4
213	21.5	10 WF 2I	9.9	3720	121.1	14 WF 78	30.7	105	16.2	10 LB 17	6.5	751	64.2	15 I 50.0	11.7
324	26.4	10 WF 25	12.3	3860	107.1	12 WF 79	36.0	122	21.4	12 LB 19	5.7	760	50.3	12 I 50.0	15.1
339	20.8	8 WF 24	16.3	3950	197.6	21 WF 96	20.0	131	10.1	6 LB 16	13.1	1020	88.4	18 I 54.7	11.5
372	34.1	12 WF 27	10.9	4010	299.2	30 WF 108	13.4	145	18.8	10 LB 19	7.7	1220	101.9	18 I 70.0	12.0
389	41.8	14 WF 30	9.3	4070	266.3	27 WF 102	15.3	175	25.3	12 LB 22	6.9	1440	116.9	20 I 65.4	12.3
437	30.8	10 WF 29	14.2	4270	151.3	16 WF 88	28.2					1590	126.3	20 I 75.0	12.6
457	24.3	8 WF 28	18.8	4310	130.9	14 WF 84	32.9	JOISTS				2210	173.9	24 I 79.9	12.7
493	39.4	12 WF 31	12.5	4490	115.7	12 WF 85	38.8	33	5.1	6×4 J 8-½	6.5	2400	185.8	24 I 90.0	12.9
529	48.5	14 WF 34	10.9	4830	248.9	24 WF 100	19.4	40	7.8	8×4 J 10	5.1	2430	150.2	20 I 85.0	16.2
529	56.3	16 WF 36	9.4	4930	138.1	14 WF 87	35.7	43	10.5	10×4 J 11-½	4.1	2590	197.6	24 I 100.0	13.1
592	27.4	8 WF 31	21.6	4890	327.9	30 WF 116	14.9	55	14.8	12×4 J 14	3.7	2640	160.0	20 I 95.0	16.5
616	35.0	10 WF 33	17.6	4960	184.4	18 WF 96	26.9					4240	234.3	24 I 105.9	18.1
665	45.9	12 WF 36	14.5	5130	166.1	16 WF 96	30.9					4640	250.9	24 I 120.0	18.5
671	54.6	14 WF 38	12.3	5150	299.2	27 WF 114	17.2	STANDARD MILL BEAMS							
708	64.4	16 WF 40	11.0	5740	354.6	30 WF 124	16.2	144	14.0	8 M 17	10.3	STANDARD CHANNELS†			
$\boxed{759}$	31.1	8 WF 35	24.4	5800	150.6	14 WF 95	38.5	159	15.2	8 M 20	10.5	7	1.1	3 [4.1	6.4
890	$\boxed{72.4}$	16 WF 45	12.3	5870	274.4	24 WF 110	21.4	236	21.7	10 M 21	10.9	8	1.2	3 [5.0	6.8
895	42.2	10 WF 39	21.2	5900	202.2	18 WF 105	29.2	262	23.6	10 M 25	11.1	10	1.4	3 [6.0	7.2
898	51.9	12 WF 40	17.3	6030	404.8	33 WF 130	14.9	320	21.0	8 M 24	15.2	11	1.9	4 [5.6	5.9
966	62.7	14 WF 43	15.4	6610	379.7	30 WF 132	17.4	350	22.5	8 M 28	15.6	15	2.3	4 [7.25	6.4
1060	89.0	18 WF 50	11.8	6660	249.6	21 WF 112	26.7					17	3.0	5 [6.7	5.6
1100	80.7	16 WF 50	13.6	6910	299.1	24 WF 120	23.1					21	3.5	5 [9.0	6.0
1120	58.2	12 WF 45	19.2	6960	220.1	18 WF 114	31.6	JUNIOR BEAMS				24	4.3	6 [8.2	5.5
1200	49.1	10 WF 45	24.5	7420	446.8	33 WF 141	16.6	7	2.4	6 JR 4.4	2.9	29	5.0	6 [10.5	5.8
1210	70.2	14 WF 48	17.2	7900	502.9	36 WF 150	15.7	10	3.5	7 JR 5.5	2.8	33	6.0	7 [9.8	5.5
1290	98.2	18 WF 55	13.1	8610	284.1	21 WF 127	30.3	13	4.7	8 JR 6.5	2.7	36	5.8	6 [13.0	6.2
1370	64.7	12 WF 50	21.2	8570	330.7	24 WF 130	25.9	14	5.8	9 JR 7.5	2.5	39	$\boxed{6.9}$	7 [12.25	5.7
14810	77.8	14 WF 53	19.0	8850	486.4	33 WF 152	18.2	20	7.8	10 JR 9.0	2.5	45	8.1	8 [11.5	5.5
1520	54.6	10 WF 49	27.9	9200	541.0	36 WF 160	17.0	23	9.6	11 JR 10.3	2.4	46	7.7	7 [14.75	6.0
1530	126.4	21 WF 62	12.1	10230	302.9	27 WF 145	25.4	38	12.0	12 JR 11.8	3.2	51	9.0	8 [13.75	5.7
1550	107.8	18 WF 60	14.4	10600	579.1	36 WF 170	18.3					59	10.5	9 [13.4	5.6
1620	94.1	16 WF 58	17.2	10660	317.2	21 WF 142	33.6	JUNIOR CHANNELS†				64	11.3	9 [15.0	5.7
1690	70.7	12 WF S3	23.9	10880	372.5	24 WF 145	29.2	10	4.4	10 [6.5	2.3	67	10.9	8 [18.75	6.2
1850	60.4	10 WF S4	30.6	12180	621.2	36 WF 182	19.6	12	6.5	10 [8.4	1.9	$\boxed{76}$	13.4	10 [15.3	5.7
1880	139.9	21 WF 68	13.4	12360	444.5	27 WF 160	27.8	18	9.3	12 [10.6	2.0	82	13.5	9 [20.0	6.1
1950	117.0	18 WF 64	16.7	13360	413.5	24 WF 160	32.3					94	15.7	10 [20.0	6.0
1980	104.2	16 WF 64	19.0	13870	663.6	36 WF 194	20.9					114	18.1	10 [25.0	6.1
2050	78.1	12 WF 58	26.3	14100	528.2	30 WF 172	26.7	STANDARD I-BEAMS				131	21.4	12 [20.7	6. 1
2130	92.2	14 WF 61	23.1	15110	492.8	27 WF 177	30.7	17	1.7	3 I 5.7	10.1	136	20.6	10 [30.0	6.6
2170	150.7	21 WF 73	14.4	17340	586.1	30 WF 190	29.6	21	1.9	3 I 7.5	10.8	153	23.9	12 [25.0	6.4
2250	175.4	24 WF 76	12.8	18410	669.6	33 WF 200	27.5	29	3.0	4 I 7.7	9.7	178	26.9	12 [3.0	6.6
2260	67.1	10 WF 60	33.6	21250	649.9	30 WF 210	32.7	34	3.3	4 I 9.5	10.2	309	41.7	15 [33.9	7.4
2330	128.2	18 WF 70	18.2	22440	740.6	33 WF 220	30.3	47	4.8	5 I 10.0	9.8	351	46.2	15 [40.0	7.6
2430	115.9	16 WF 71	21.0	24150	835.5	36 WF 230	28.9	64	6.0	5 I 14.75	10.7	421	61.0	18 [42.7	6.9
2630	88.0	12 WF 65	29.9	26850	811.1	33 WF 240	33.1	73	7.3	6 I 12.5	10.0	429	53.6	15 [50.0	8.0

(*continued*)

TABLE 7B.1.5 Structural Steel (*continued*)

SL.	*S*	Section	L.	SL.	*S*	Section	L.	SL.	*S*	Section	L.	SL.	*S*	Section	L.
2640	103.0	14 WF 68	25.6	27540	892.5	36 WF 245	30.9	93	8.7	6 I 17.25	10.7	440	63.7	18 [45.8	6.9
2690	73.7	10 WF 66	36.5	31290	951.1	36 WF 260	32.9	106	10.4	7 I 15.3	10.2	491	69.1	18 [51.9	7.1
2830	196.3	24 WP 84	14.4	36810	1013.2	36 WF 280	35.7	130	12.0	7 I 20.0	10.8	544	74 .5	18 [58.0	7.3
2850	141.7	18 WF 77	20.1	42180	1105.1	36 WF 300	38.2	151	14.2	8 I 18.4	10.6				
2870	168.0	21 WF 82	17.1					178	16.0	8 I 23.0	11.1				
2940	127.8	16 WF 78	23.0			LIGHT BEAMS		278	24.4	10 I 25.4	11.4				
3120	112.3	14 WF 74	27.8	62	9.9	8 LB 13	6.3	353	29.2	10 I 35.0	12.1				
3210	97.5	12 WF 72	32.9	67	7.2	6 LB 12	9.3	407	36.0	12 I 31.8	11.3				

*Data from *Eng. News Record*, March 18, 1948. Article by William P. Stewart.
†These sections should also be investigated for torsion.

process could be utilized to select structural steel of other design. The example given in the Table 7B.1.4 is for wide flange beams.

7B.1.9 Rotted Substructure

As stated earlier, the greatest cost in replacing existing wood floor supports is *access* (see 7B.1.5). Once the access problem is addressed, removal and replacement of the wood supports or "stiff-legs" is handled as discussed in Section 7B.1.3. Any rotted wood must be replaced. Prudent practice suggests the replacement of *all* supports (perimeter and exterior) once any leveling action is initiated.

*7B.1.9.1 Causes of Wood Deterioration**

One of the most common causes of the degradation of building timbers is decay of the wood structure caused by wood inhabiting fungi. This infection of the wood frequently alters its physical and chemical characteristics. The extent of the deterioration depends to a large extent on the degree of the decay and the specific effects of the organism producing it. The strength and density of the material will be drastically reduce and the color will be permanently modified.

During the invasion stage, there is frequently no visible change in wood other than the possible discoloration. Often, the condition could go undetected for a period of time. In the late or advanced stages of decay, the wood may become punky, soft and spongy, stringy, pitted, or crumbly depending on the nature of the attacking fungus and time lapsed. The development and growth of wood inhabiting fungi requires moisture, air, oxygen and relatively warm ambient temperatures. Decay is most prevalent in wood that is either in direct contact with damp ground or located where moisture collects and cannot be readily evaporated. This emphasizes the need for proper ventilation in the crawl space to eliminate moisture. Although individual fungi show a variation in the precise moisture requirements, it is recognized that a moisture content above the fiber saturation point (25 to 30%) is required for optimum development. A moisture content below 15% (dry) will completely inhibit growth. Moreover, if wood in which decay has already started is dried to a moisture content below 20%, the development of the fungus will be stopped. However, once this infected wood is again placed in an environment creating a moisture content in excess of 20%, decay will generally start again.

Dry rot (or the last stages of brown rot) describes the condition when wood becomes brittle and crumbly. This is usually more obvious as the wood dries. The name dry rot is a complete misnomer. No wood will decay while it is dry. The dry rot fungus, which frequently is found growing in com-

*Special appreciation goes to Dr. Don Smith, Biology Department, University of North Texas, for input to this section.

paratively dry locations, is capable of seeking and supplying its own moisture through root-like strands of mycelium. Timbers in touch with moist ground can often provide moisture for decaying areas 15 to 20 ft. away. Many engineers suggest wood replacement when the structural timber (sills, joists, girders) shows more than 20% damage.

Sometimes mold, mildew, and fungus stains are present on building timber surfaces. Most molds or fungus stains have little effect on the structural integrity of the timber. They generally indicate surface conditions that are also conducive to the growth of wood destroying fungi. However, structural problems related to the invasion of mold and mildew are less preponderant than those created fungi.

Most wood that has been wet for any considerable length of time probably will contain bacteria. The presence of a sour odor manifests bacterial action. Usually, the greatest effect of cell-dissolving bacteria is that it allows for excessive water absorption, which can promote strength loss in some species of wood. This occurrence is somewhat similar to that described for mold and mildew, in that the frequency of serious problems is somewhat limited.

In summary, the decay of timber leading to a structural deficiency is generally termed wood rot. These conditions result most often from the development of a wood destroying fungus that feeds on the wood, affecting its strength and density. This condition normally occurs over a long period of time. Frequently, there are periods when the fungus is dormant, and growth and decay are stopped due to ambient temperature and environmental changes. Building conditions that can promote the growth of fungus and the resulting decay of the wood timber include such factors as poor drainage (which results in moist soil), wood in contact with moist soil, long-standing plumbing leaks, wood not properly treated or protected, and poor or improper ventilation. Any act that prevents or removes moisture is a preventative measure.

Prudent practice suggests the replacement of *all* supports (stiff legs) (perimeter and exterior) once any leveling action is initiated. HUD encourages this practice.

7B.1.9.2 Costs

The costs to replace wood member vary from application to application. However, a rough cost estimate for removal and replacements would be as follows:

1. Joists/girders (2″ × 6″ or 2″ × 8″)	$25/board foot
2. Sill plates (2″ × 4″)	$23.00/linear foot
3. Pier caps	See Section 7B.1.3

The cost basis (labor, etc.) would be the same as that specified in the preceding. Refer also to Section 7B.1.3, which gives costs for some installations.

7B.1.10 Precautions and Prevention

Section 7A presents a detailed discussion of measures for avoiding or minimizing foundation failure. The ensuing paragraphs will, however, highlight concerns particularly of interest to pier-and-beam foundations.

7B.1.10.1 Drainage and Ventilation

It is most important to prevent standing water in the crawl space, even if the accumulation is sporadic. Occasionally, construction practices (such as the low-profile pier-and-beam foundations) tend to promote this problem. The crawl space is lower than the outside grade. Since water always seeks the lowest level, there exists a natural attraction. Water in the crawl space creates several conditions that are unacceptable. These include: 1) an attraction for termites, 2) the build-up of mold and fungus, 3) unpleasant odor, 4) wood rot, and 5) potential foundation damage. Attempts have been made to "mask" certain of these problems by covering the crawl space with polyethylene. This could have some beneficial influence on numbers two (2) through four (4) but might actually increase the likelihood of (1) and (5). The best cure is to: 1) establish proper drainage away from the perimeter so water flows away from the structure and 2) provide sufficient ventilation so any intruding moisture will be promptly removed. The latter might require a forced air blower to increase circulation and/or installation of additional foundation vents. (Building codes suggest one square foot (0.9 m^2) of vent per 150 ft^2 (13.5 m^2) of floor space). Flower boxes and curbing can also create sources for water. Flower boxes should have a concrete bottom and drains that direct excess water to the exterior, away from the perimeter beam. Flower bed curbing must also contain provisions to drain water away from the perimeter beam.

7B.1.10.2 Crawl Space

As stated above, the crawl space should be dry and provide access to the foundation, utility lines, ducts, etc. As a rule, the clearance should be at least 18″ (0.45 m). Adequate ventilation should also be a primary concern. Most building codes suggest one square foot (0.09 m^2) of air vent per 150 ft^2 (13.5 m^2) floor space.

7B.2. MUDJACKING (SLAB)

7B.2.1 Introduction

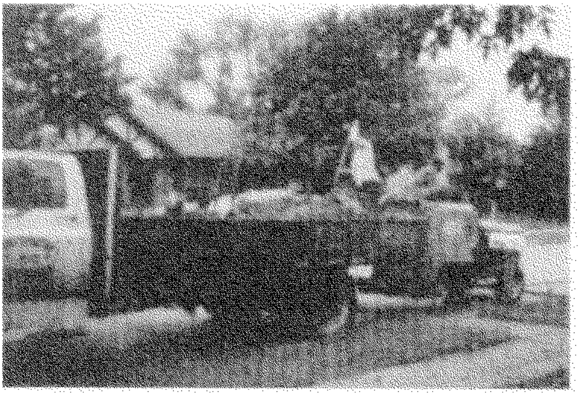
Slab foundations suffer the problems of both settlement and upheaval. The general approach to the perimeter is much the same as that described in Section 7B.4. However, due to design, mudjacking is required to fill voids and raise and stabilize the foundation.[15–17,19,23,36,60,79,86,102]

Routine mudjacking represents an areas where the contractor is generally denied any margin of safety. As voids are filled and the foundation raised, pumping must cease when the displaced segment of the foundation slab has been restored to desired grade. About the only factor in control of the contractor is to attempt to assure as near complete filling of voids as possible—to supply as close to 100% foundation bearing support as possible. This is discussed in the following paragraphs. Specific sit compromises often preclude as thorough filling as might be otherwise desired. For example, it is often decided not to drill holes through ceramic or sheet linoleum floor tile and sometimes the location of utility lines beneath the foundation also interfere with mudjacking. Subject to these restrictions, mudjacking is often not warranted by the contractor. When coverage is provided, it is often limited to 12 months. Flatwork (which has no perimeter beam) is often performed with no limited warranty. Recurrent settlement of foundations properly mudjacked is not too common, less than 1 to 2%. Comparatively, flatwork is about 10 times as likely to resettle, often due to erosion of soil beneath the placed grout. "Flatwork" includes such installations as walks, patio, drives, streets, parking slabs, and pool decking.

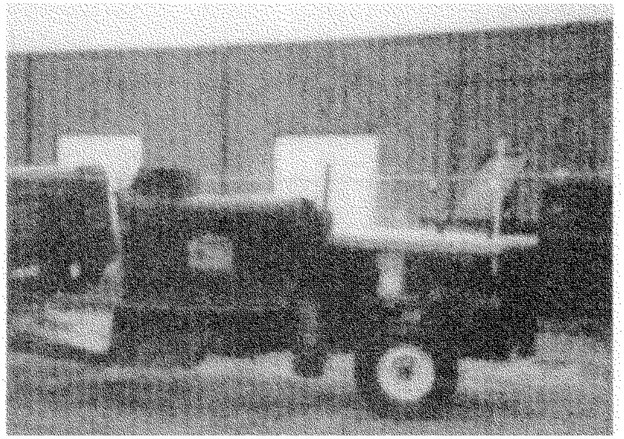
For all practical purposes, conventional deformed bar and posttensioned slab foundatons are treated as the same. In the latter case, it sometimes becomes necessary to also repair or retension defective cables.

7B.2.2 Equipment Required

Mudjacking is a process whereby a water and soil–cement or soil–lime–cement grout is pumped beneath the slab, under pressure, to produce a lifting force that literally floats the slab to the desired position. Figure 7B.2.1 depicts the normal equipment required to mix and pump the grout. The mudjack shown behind the truck is a Koehring Model 50, theoretically capable of pump rates to 10 yd^3/h (8 m^3/h) and pressures to 250 lb/in^2 (1725 kPa). The grout is introduced via small holes drilled through the concrete. The rubber nozzle affixed to the end of the injection hose serves as a packer to contain the grout beneath the slab. Refer to Figure 7B.2.2. The pattern for holes drilled to facilitate grout injection is dictated by the specific job condition and guided by experience. "Down" holes are



(a)



(b)

FIGURE 7B.2.1. (a) Mudjacking equipment; (b) close-up view of Koehring Model 50 Mudjack.

drilled vertically through the slab surface and used to conditionally raise interior areas of the slab. These holes are located to both avoid unnecessary floor damage and, at the same time, provide the best possible results. Ceramic tile and sheet linoleum floor covering often influence the hole pattern. Other holes are drilled horizontally through the perimeter beam. Where possible, these holes are drilled below grade. The back fill then covers their presence.

Often, routine mudjacking is mistakenly referred to as pressure grouting. Refer to Section 7B.3. The weight of a typical 4 in thick (10cm) concrete slab is on the order of 50 lb/ft^2 (810 kg/m^2). In terms of pressure, this relates to less than 0.35 lb/in^2 (2.4kPa). Mudjacking the perimeter beam where applicable would require greater pressure. Neglecting breakaway friction, this load could approach 5 lb/in^2 (34 kPh). In either event, "high pressure" is hardly a descriptive a term.

Seldom is any uniform grid applicable except in routine raising of open slabs such as pavements, walks, floating slabs (i.e., warehouse floors) or aspects of pressure grouting (See Section 7B.3). In

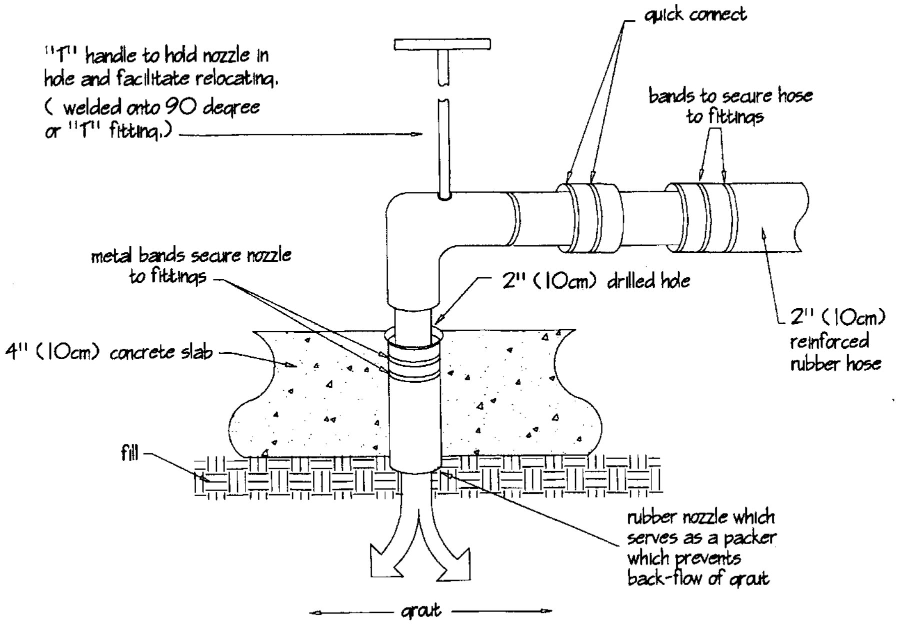


FIGURE 7B.2.2. Nozzle pack-off.

fact, often during slab foundation leveling, the hole pattern is adjusted to provide the desired control and results. The floor pattern is controlled by selective use of injection bleed holes. For example, pumping starts at hole "A" (usually the lowest area). Holes surrounding the injection site are left open. As pumping continues (to fill voids) the grout will appear at one or more of the surrounding holes. In order to direct the grout to a different location, the bleeding hole(s) is temporarily plugged. Tapered, round wood pegs or rolled up cement bags can be used as plugs. When voids are filled, all holes except the injection site are plugged. Continued pumping produces the selective raising. It is important to limit the raise at each hole to perhaps ¼ to ½ in (6.3 to 12.6 mm). Mudjacking gradually proceeds in a more or less circular pattern to the extremities. Upon completion, the mudjack holes are patched with a low slump of concrete. The patches are raked off even with the slab surface.

7B.2.3 Materials and Composition

Grout composition and consistency is an important consideration. A typical grout (28 day compressive strength of 50–100 psi or 345–690 kPa) could consist of two sacks of cement, [188 lb (85 kg)], 1800 lb (820 kg) siliceous soil, and 70 gal water per cubic yard (330 l/m^3) of dry mix. Due to outside, open storage, the soil is usually somewhat wet. A very dry soil may require additional water for ideal consistency.

The soil should be free fo roots, aggregates or other bulky materials that might clog either the pump or the lines and should contain little or no clay. As the clay content increases, so do mixing and handling problems. A simple procedure that may help screen a procpective soil for use as a grout base follows.

Place 1½ cups of the prospective soil into a quart (0.95 L) fruit jar. Fill with water and add cap. Shake briskly until sample slakes (no lumps). Set the jar on a level surface. Time and measure the rate of settlement of each layer. Gravel pebbles and coarse sand (less than ¼″ (6.4 mm) in diamter) will settle within a matter of seconds and should constitute between 0 and 10% of the sample. The fine sand components will settle within 15 to 30 seconds and should account for 20–30% of the sample. The clay faction will require several hours to settle and should account for 0–10% of the sample.

The rate of settlement can be calculated for various materials by using the Stokes' equation. Refer to Section 2A.3.

The two sack mix is the choice for general mudjacking. The weight of cement to soil ratio is 10% (188/1800). Accordingly, the ratio for a three, four, or five bag mix is 15%, 20% and 25%. Unless the grout is watered down, this amount of cement provides adequate strength, limits shrinkage, and facilitates flow.

The set grout is also friendly to excavation or re-entry.

Depending upon specific cases, richer grouts can be used. For example, in cases requiring increased strength, a four sack grout might be used [compressive strength approximately equal to about 400 psi (2760 kPa) at 28 days]. In cases of dam grouting, the grout used might be nothing more than cement and water (5 gal/sack), often referred to as a neat cement grout (compressive strength approximately 8295 psi after 28 days). Consistency is normally varied by adjusting the solids to water ratio. A thinner grout will migrate over a larger area, sometimes allowing a greater lifting force at the same pump pressure (F = Pa). A think grout is more prone to bleed out from the work area. A thicker grout (with less flow) will be restricted to a smaller area. A substantial lifting force may be possible as a result of increased, confined pressure. In some instances, the thicker grout still tends to escape from the work area. Consider, for example, that situation where the grout flows beyond the foundation or intended work area. This could represent a project involving an interior fireplace (concentrated load) surrounded by a normal slab (low weight and strength), a shallow (or absent) perimeter beam, or the like. If the thickened grout does not provide the solution, other options in order of increasing difficulty (and expense) are:

1. *Stage Pumping*. This practice involves the placement of a volume of grout followed by a period of shutdown sufficient for the in-place grout to thicken or perhaps reach initial set. The process is then repeated. Care must be given to limit the shutdown time to prevent the grout f from setting-up in the hose. Refer also to Section 7B.2.4.
2. *Shoring for Containment*. Sheet piling (plywood or suitable material) is driven or buried into the soil at the slab perimeter and shored by suitable bracing. Under most conditions the seal between the sheet piling and concrete perimeter is sufficient to contain the grout.
3. *Underpinning*. Underpinning can be used to raise the slab perimeter, followed by mudjacking to fill voids. The underpinning supports the structural load, removing resistance to grout flow. On occasion, sheet piling might again be utilized to further contain the grout.

7B.2.3.1 Variations in Grout Mix

More complicated grout mixes are sometimes specified mudjacking of slabs such as 1) highways, 2) runways, 3) parking lots, or 4) upon rare occasion, commerical or even residential foundations. These mixtures often involve constituents such as fly ash (pozzolan), cement, sand, siliceous soil, surfactant (to reduce surface tension or water requirements), lime, bentonite, (montmorillonite), sodium chloride, and water.

NOTE: Proprietary grouts are also specified on *vary rare occasions*, due largely to cost and handling problems. These include such products as : Polyurethane (Uretrek), Cemill (microfine cement), polyacriylomides, and sodium silicates. Expense and limited improvement over conventional mixes inhibit any wide acceptance of complex mixes, at least for mudjacking as defined herein. Various grouting operations, however, frequently use the special mixes/products. See references 8, 17, and 87 and Section 7B.3.7.

Fly ash or pozzolan has been utilized as a companion to cement for centuries. In fact the early aqueducts constructed by the Romans (and existing in part to this day) were made from a cementitious material consisting of pozzolanic earth, $Ca(SO_4)$ and water. Generally the fly ash is intended to reduce 1) the cost of the cement grout with minimal loss in strength, 2) the grout unit weight, and 3) in some cases, grout shrinkage. The principal disadvantages to the use of this produce in mudjacking relates to its very abrasive effect on pump and mixing equipment and the practically nonexistent cost savings. Fly ash–cement grouts can, in fact, be much more expensive than conventional grout, depending upon the volume of siliceous soil replaced by fly ash.

Surfactants generally do not serve any real purpose in mudjacking. The grout is placed essentially through voids (no permeability problem), and consistency (flocculation) can be adequately controlled by the discriminate use of water. Lime is sometimes useful to modif the properties of the grout. However, the contact of lime and sulfates in clay soils can be quite deleterious. See references 17, 48, 87, and 85.

Bentonite has limited use in mudjacking. Principally, this is used either as an additive to make harsh solids pumpable (i.e., fly ash or sand) or to reduce friction (see Section 7B.3.7.2). Bentonite can be also preswelled to create volume. Then, upon mixing with cement, the product will develop a "set." Saline, water (containing sodium chloride or potassium chloride) will serve to reduce the amount of bentonite swell. This reduces the volume increase and tends to somewhat enhance strength. However, if final strength of the grout product is a concern, special care should be given to both the application and composition of bentonite grouts.

7B.2.4 Grouting Pressure

Grout placement pressure is another concern. Most often this aspect is overrated because foundation slabs represent minimal loads. For example, the weight of a 4 in thick (10 cm) slab is about 50 lb/ft^2 or 0.35 lb/in^2 (244 kg/m^2).In most residential construction, the added live load plus dead load on the interior slab is minimal, often assumed to be 60 lb/ft^2 (292 kg/m^2). The perimeter loads are substantially greater, perhaps 600 to 800 lb/lin ft (890 to 1190 kg/m). However, the perimeters are seldom intended to be raised solely by mudjacking. (This was not always the rule, but expediency and lack of gifted operators has prompted the compromise.) Mutiple-story construction increases the weight, but still the loads are relatively low. Hence, most mudjacking is accomplished at very low pressure. Significant pump pressure surges are generally the result of various forms of friction increase—plugged lines, improper grout mixture or, upon occasion, that instance where the member to be raised suffers some form of mechanical binding or resistance. Once the grout leaves the pump, there are few instances that could account for a measurable pressure drop. The most frequent cause is a parted hose or blowout. This is best comprehended by understanding the minimal resistance offered by the slab during mudjacking procedures and the principals involved. The force (F) developed by the grout that is capable of lifting the slab is represented as $F = P_D \times A$, where P is the delivered grout pressure in psi and A is the area over which the pressure is applied in in^2. The units of F would be in pounds. Increasing *either* P_D or A proportionately increases F. The relationship between P and the observed gauge pressure (P_G) is $P_G = P_D - P_f$, where P_f is the friction loss (also referred to as line friction). Refer also to the sections on grouting—7B.3 and in particular 6B.

Be very alert about any pumping stoppage. During *routine* mudjacking these stoppages are only long enough to permit moving the nozzle from hole to hole. The duration is short and since this process more frequently occurs during the void filling phase, the problems are minimal. The pause in grout movement through the hose is subject to several factors that complicate, or in worst case scenarios, completely prevent continuation of pumping. The factors that contribute to this are:

1. The dilatant nature of the grout. With dilatant fluids, the apparent viscosity is proportional to the applied stress. When in motion, the apparent viscosity is relatively low. When motion stops, viscosity increases (due to stress) and the grout behaves somewhat as a solid. In this state, a disproportionately large force is required to reinitiate movement. The lower the water to solids ratio, the more pronounced this problem becomes.
2. Cohesion tends to return to the clay, again increasing apparent viscosity.
3. Some cementitious reactions develop between the siliceous sand and cement. Temperature and time are the allies for this reaction (along with the amount of cement and water in the grout).

In possible difficult situations, it is advisable to clear the pump hose prior to any shutdown. This can be done by purging the grout in the lines. The options are wasting the grout or circulating the grout back into the mixer. Water is most often used to clear the lines. If situations demand, the lines can be cleared by placing a sponge in the line (at the discharge of the pump) and pumping the sponge through the hose using water. *Do not get in front of the hose*. The sponge might be ejected at a high velocity. When air is used to displace the sponge, even greater care must be exercised.

7B.2.5 Grout Volume

More significant than placement pressure is the placed volume, since this aspect has a direct bearing on project cost. In normal slab-raising operations, an operator who is capable of mixing and pumping on the fly can place up to about 16 yd^3 (12.8m^3) per 8 hour day. For most mudjacking (and grouting) operations, the grout volumes are in "wet" cubic yards, which correlate to the calculated void volume.Wet volume is computed by including the water.For example, the typical grout mix would contain 1 yd^3 soil (0.76 m^3), 2 ft^3 (0.57 m^3) cement, and 9.6 ft^3 (72 gal) of water. This would provide a yield of 1.4 yd^3 of grout per yd^3 of soil. The complexities imposed by foundation leveling reduce this capacity to a maximum of about 10 yd^3 (8m^3), which corresponds to a volume of about 800 ft^2 (74m^2) in area and 4 in (10 cm) thick. Void filling (amounting to open-ended pumping) can sometimes reach a placement volume of 20 yd^3 (16m^3) or so per 8 hr day, if material supply and handling can support the quantity. If placement volumes are required in excess of these, the best solution is multiple pumps. (For other grouting operations, greater capacities are possible). The material can sometimes be batched and fed to alternative pumps, capable of pumping 25 to 30 yd^3 (19 to 23 m^3) per hour or 200 to 240 yd^3 (160 to 190m^3) per 8 hour day. Refer again to Section 7B.3. Neglecting slowdowns imposed by changing injection sites, allowance for set time, materials supply, etc., the placement limitation (imposed solely by pressure) can be explained by the relationship.

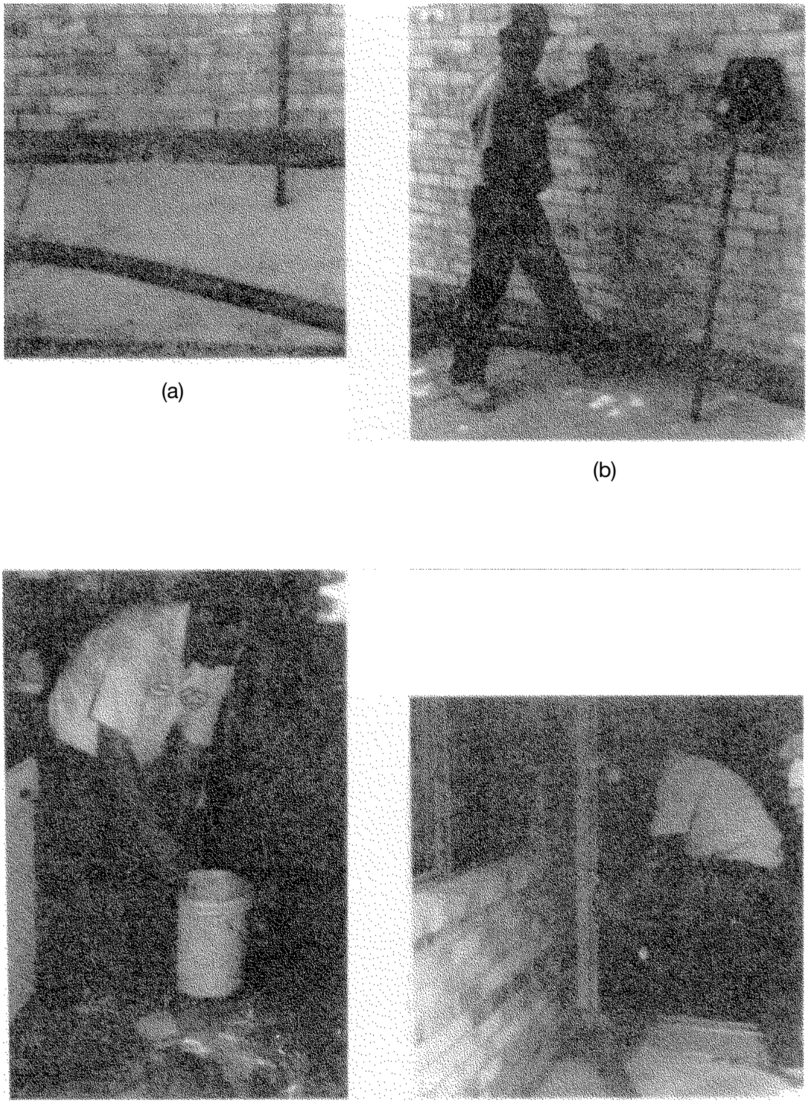
$$HHP = kBHP = \frac{Q \times P}{7.2}$$

where HHP = hydraulic horsepower
k = an efficiency constant (frequently 0.75 or less)
BHP = brake horsepower
Q = rate of flow, ft^2/min
P = pressure, lb/in^2

At maximum conditions, this equation shows that, as the placement pressure increases, the volume pumped decreases proportionately. Note that the placement pressure is the resistance at the oump cylinder head and not the pressure resistance offered by the member being jacked. Friction pressures developed during normal mudjacking operations far exceed the load resistance required in slab raising. In fact, something in excess of about 150 to 200 ft (45 to 60 m) of 2 in ID (5 cm) pump hose and a thick grout will produce a friction pressure approaching the capacity of the average Koehring machine (see Table 7B.3.4). Figure 7B.2.3 illustrates the mudjacking process.

7B.2.6 Mudjacking to Level a Slab Foundations

In leveling a slab foundation the primary concerns are: 1) avoiding unnecessary damage to floor covering (generally every attempt is made to limit or eliminate the need for drilling holes through ceramic tile or sheet linoleum), 2) raising the areas that are below the desired grade, 3) ascertaining that all voids are properly grouted, and 4) rendering the foundation to as near as-built condition as practical without creating undue, additional damage to the structure. This is accomplished by selective injection through previously drilled holes.



(a) (b) (c) (d)

FIGURE 7B.2.3. Mudjacking a slab. (a) Injection hole drilled through perimeter beam. (b) Nozzle in place and mudjacking in progress. (c) Interior pumping. (d) Exterior pumping of patio slab.

The drilling pattern and injection hole selection depend largely on the specific project and intent. Prior to grout injection, all the holes within the proposed work area should be drilled. These holes can then be selectively pumped to accomplish the goal result. Open holes adjacent to the one being injected are utilized to: 1) establish the grout flow pattern and, 2) create a safety valve to prevent unwanted raise. As the grout spreads, holes are alternately plugged to control grout flow and facilitate the raise. A raise is practical only in areas of either injection or plugged holes. (Temporary plugs can be anything from tapered wood plugs to rolled-up cement sacks.) Upon occasion, performance might suggest the need for additional strategic injection holes. These are drilled on an as-needed basis. Experience will enable a competent mechanic to raise most any slab foundation satisfactorily and safely. On the other hand, narrow flat work poured on top of the grout is not representative of responsible mudjack projects. Other circumstances that often introduce concern or difficulty include:

1. Concrete already broken and cracked into relatively small sections
2. The foregoing without reinforcing
3. Flatwork with no perimeter beam or thickened edge. (Often, this problem can be remedied by using plywood or steel sheets to provide a block to contain the grout.)
4. Heavily loaded sections situated between normal concrete slabs. (An example of this could be an interior fireplace, a firewall between two living areas, or load-bearing perimeter beams.)
5. Situations where a floating slab is dowelled into a perimeter

Where mudjacking is not feasible, the usual option is to break out and repour the affected area.

Grout control is essential for producing a successful raise. In addition to hole selection, another useful approach for controlling grout flow and confinement is to vary the grout consistency or viscosity. A thin grout will flow (create a greater area of influence) and the thicker grout will restrict flow (facilitate a greater lifting force.) Refer to Section 7B2.5. Occasionally, some form of mechanical confinement might also be applicable.

Sometimes, job-specific conditions might tend to impede proper grout performance. In the case of concentrated loads, where conventional methods fail to accomplish the desired results, several options can be considered. The first and simplest option is to attempt stage grouting. Refer to Section 7B.2.3. Another alternative is to pump two or more holes simultaneously to increase the areas over which the grout pressure is applied. Refer to Section 7B6.9. The least attractive option is to underpin the loaded area using access provided by breaking holes through the floor slab. For the record, this about the only situation where underpinning through a floor slab is acceptable.

7B.2.6.1 Settlement

Slab settlement restoration is a relatively straightforward problem. Here the lower sections are merely raised to meet the original grade, thus completely and truly restoring the foundation. The raising is accomplished by the mudjack method, as described above. In some instances where concentrated loads are located on an outside beam, the mudjacking may be augmented by mechanical jacking (installation of spread footings or other underpinning). In no instance should an attempt be made to level a slab foundation by mechanical means alone. Instead of providing interior support (and stabilizing the subsoil), mechanical raising creates voids which, if neglected, may cause more problems than originally existed. As a rule, residential foundation slabs are not designed to be bridging members and should not exist unsupported. (Mechanical techniques normally make no contribution toward correcting interior slab settlement.) Raising the slab beam mechanically and back-filling with grout represents a certain improvement over mechanical methods alone, but still leaves much to be desired. This approach loses the benefits of "pressure" injection normally associated with the true mudjack method. The voids are not adequately filled, which prompts resettlement.

Figure 7B.2.4 illustrates the effect of leveling. In this instance, involving interior slabs, leveling was accomplished by mudjacking. Perimeters were generally leveled by underpinning techniques. In the "before" pictures, the separations under the wall partition, in the brick mortar, and between brick and door frame, and the settlement of the floor slab are obvious indications of foundation distress. In the "after" pictures the separations are completely closed, illustrating that the movement has been reversed or corrected. In this particular example, the leveling operation produced nearly perfect restoration. This is not always the case, as discussed in various sections of this book. The remaining photos in this section depict the mudjacking process in action.

7B2.6.2 Filling Voids

Mudjacking is used in a number of other applications either as a remedial or a preventative tool. Section 7B.3 covers some of the more spectacular of these. However there is always the more mun-

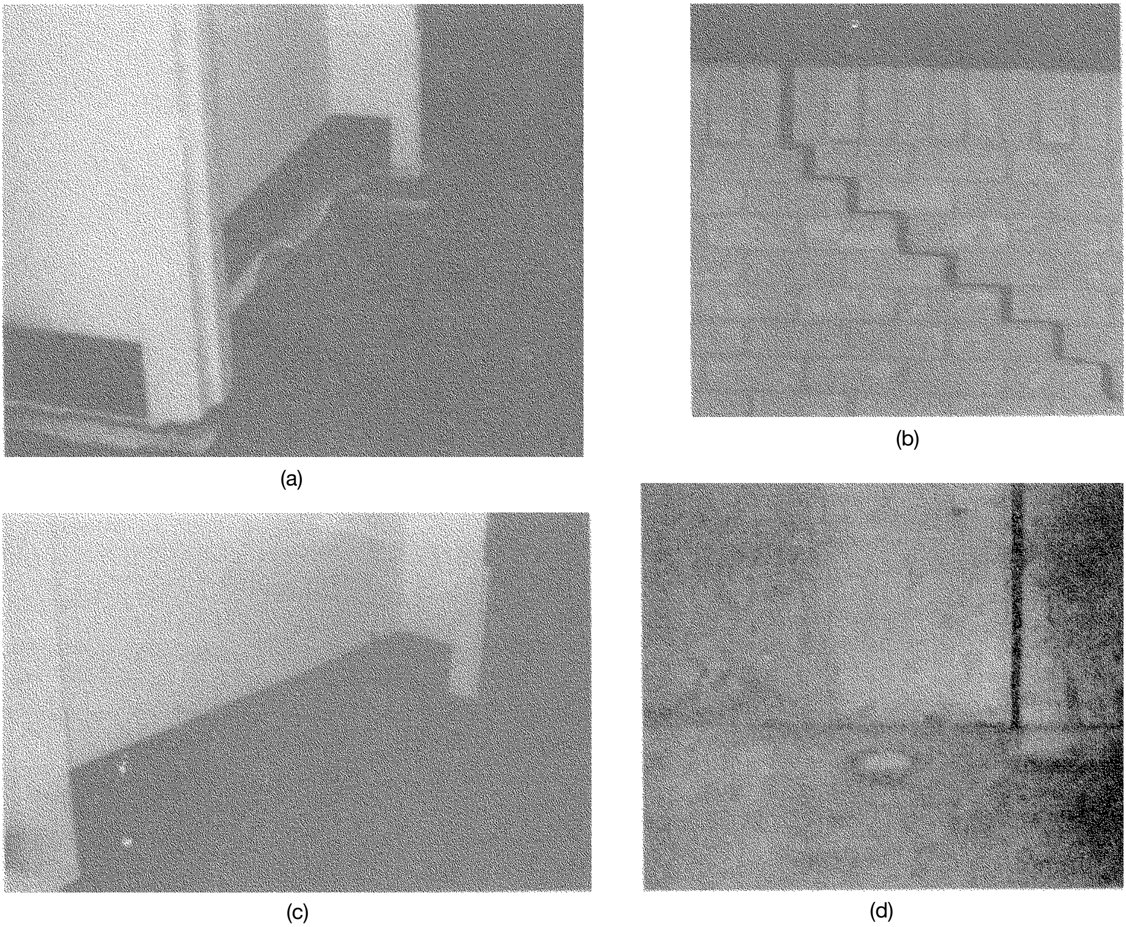


FIGURE 7B.2.4. Foundation Leveling. Before: (a) The floor is separated from the wall partition by about 4 in (10 cim); (b) the separation in brick mortar is in excess of 2 in. (5 cm). After: (c, d) In both instances the separations are completely closed. The end results of foundation leveling are not always so impressive.

dane. This could involve such projects as filling voids to prevent problems. Examples of this could include such projects as filling voids created by water erosion of fill: 1) beneath foundations, 2) around utility lines, 3) outside tunnels, and 4) beneath pavement. Another application is back-filling excavations created to provide access to underground utilities.

7B.2.6.3 Sewer Lines

Questions frequently surface regarding the interaction between mudjacking and sewer lines. These questions generally boil down to “did the mudjacking cause or contribute to the sewer damage”? Another issue is whether underpinning contributed to or caused the plumbing damage. Among incurance companies, a defense has been to claim that foundation repair companies cause sewer leaks, which then affect foundations. This position is weak form the standpoint of pure logic. First, the repair contractor would not be on the site unless a foundation problem already existed. Second, nothing moves unless forced to do so. With residential foundations on expansive soils, this force is most often water, either natural or (more frequently) domestic. If the basic distress is upheaval, it becomes apparent that the water already existed, long before the foundation repair contractor was called to the scene. The foundation repair contractor could hardly be accountable for causing the leak. Sometimes, during foundation leveling, plumbing damage might occur, particularly during underpinning. For this reason, probably all foundation repair contractors include a disclaimer that specifically excludes responsibility for damage to underground utilities. The frequency of this occurring is less that 1%. Those instances involving perimeter raises over 4 to 6 in (10 to 15 cm) are also sometimes susceptible to plumbing damage.

Sometimes, the allegation arises that mudjacking causes sewer damage. This is unlikely, but if it does happen: 1) the line separation would be in vertical (and not lateral) lines, and 2) the line would fill with grout during mudjacking. As a precaution, a competent contractor runs water during the time that mudjacking is ongoing in areas with plumbing. The water is intended to elutriate the cement from the grout thus preventing any cementious action. If the sewer does plug, it is quite simple to roto-root the line. This evasive action should be taken quickly; certainly before 24 hours. The grout becomes plastic/solid within about 4 hours. Beyond this time the grout does not move or flow. In 24 hours, the grout could attain a compressive strength on the order of 4000 psf (19,000 kg/m^2). If grout does enter the sewer line, it would obviously occur during pumping and not hours, days, or weeks later.

It helps in understanding the infrequency of mudjacking creating plumbing concerns when the condition of the repair is understood. First, in the event of upheaval, the slab high points are generally in the vicinity of areas with plumbing. Obviously, these locations do not then require additional raising. Second, the plumbing lines at the location of the fixtures represent the highest grade elevations within the sewer system. This, plus the fact that the lines are not covered by a large amount of back fill allows more flexibility in the lines at these points. Nonetheless, most foundation repair companies recommend a thorough utilities test before or after (sometimes both) repairs.

7B.2.7 Relative Cost

As foundation repairs go, mudjacking is one of the least expensive. A 1200 square ft (112 m^2), slab foundation with routine settlement [3 in (7.5 cm) maximum] can normally be mudjacked in a day. Based on 1999 U.S. Dollars and conditions equivalent to those established in Section 7B.1, this would cost in the range of $1400 to 2000 (per day).

Mudjacking is not intended to be a sure-all; it is intended to provide specific benefits. In truth, the charge of high costs of mudjacking per square foot is ill-founded and based on lack of understanding, misapplication, or both. As a rule, mudjacking is essential to the proper repair of conventional residential slab foundations. Foundations can be designed to cantilever or bridge void areas. However, such design practices are not common in residential and light commercial foundations. To further complicate the situation, the actual foundation design is most often not known to the foundation repair engineer or contractor.

Other costs include time and material and charges for volume pumped. The time and material is merely a breakdown of the per diem charge. The per hour charge is often in the range of $180.00 to $280.00. Generally, travel and cleanup is charged as job time. Materials are added on a cost or replacement value plus 25 to 45% extra for handling. Assuming the daily charge of $180.00 to $280.00. Generally, travel and cleanup is charged as job time. Materials are added on a cost or replacement value plus 25 to 45% extra for handling. Assuming the daily charge of $180.00 (with 8 hour minimum), the base charge would be 8 × $180/hour-$1280.00. Assuming materials cost of $400.00, the material would be charged at $400.00 × 1.25 (the handling factor) or $500.00. The charge for this hypothetical day's pumping would then be $1280.00 + $500.00 or $1780.00.

On some occasions, the job charges may be based on the cubic yard of grout pumped. A "typical: mudjack grout identified by Westco Research Labs in 1970's consisted of the following[18]:

siliceous soil =	1800 lbs	15.7 ft^3
cement (4 sacks) =	376 lbs	2.0 ft^3
H_2O =	70 gal	9.3 ft^3
(1 yd^3 soil = 3000 lb)		27.0 ft^3 or 1 yd^3
(7.48 gal H_2O = 1 cf)		
3000/1800 = 1.65		

For purposes of semantics, this mixture is sometimes referred to as a "wet yard." This "wet" volume represents the void which can be filled by a yard of grout. The cited formulations will yield 1.65 yd^3 of grout per cubic yard. (This grout would set up to produce a solid mass with a *24* hour compression strength of *6480* psf.) For composition variations, consider the following:

Correction Factors:

2 sacks cement =	1.0 ft^3
7.45 gal H_2O =	1.0 ft^3
1.0 gal H_2O =	0.134 ft^3
1 lb soil =	0.009 ft^3
111 lb soil =	1.0 ft^3

Example:

Calculate the "wet" factor for a mix containing 2 sacks cement and 80 gal H_2O:

2 sacks cement =	1.0 ft^3
80 gal H_2O, 80 × 0.134 =	10.74 ft^3
total H_2O and cement =	11.74

cubic feet = 27–11.74 or 15.26 ft^3

then weight of soil = 15.26/0.009 = 1696 lb

therefore, 1 cubic yard of soil would yield: 3000 lb/1696 lb = 1.77 yd^3 grout

Since measurement frequently monitor only the dry soil and sacks of cement used, the approximate yield factor is relied on to produce charge yards. The grout is often charged at the rate of \$5.50 to \$7.50 (for the basic formulation) per cubic yard.

Hence, assume a yield factor of 1.65 and a grout cost of \$6.00 per cubic foot. The cost to customer would then be:

$$11.65 \times 10\ yd^3 \times 27\ ft^3/yd^3 \times \$6.00 \text{ or } \$2673.00$$

The price per cubic foot of grout placed is frequently the charge basis for "deep" grouting described in 7B.3, as well as the grouting operations covered in section 6B. In the latter case, more sophisticated grout composition are used and the cost varies accordingly.

7B.3. "DEEP" GROUTING

7B.3.1 Introduction

Remedial measures to correct foundation failures can, in some instances, require more than strictly underpinning and shoring. The problems to be addressed in this section will be limited to a depth of about 30 ft. More specifically, failure of supportive elements of the foundation related to loose fill on poorly compacted subgrade materials can be corrected by densification or solidification through a process referred to as "deep" grouting or, more specifically, compaction grouting. In discussions covered by this section, the term deep could be misleading. Actually, the reference to "deep grouting" is often used to delineate grouting from mudjacking. Refer to Section 6B. The grouting covered in this section (densification) generally creates suitable and competent bearing strata, which subsequently stabilizes the structure from future failures. Grouting is particularly suitable to stabilize landfills or sanitary landfills.

Deep (or more correctly, intermediate) grouting involves the injection of a soil/cement/water grout into a loose or fissured soil subgrade (or fill). This grout is pressure injected to the extent that water and/or air is displaced, voids are filled, and the less dense material becomes encapsulated. The grout that infiltrates and encapsulates the soil cures or sets to "solidify" the subgrade. Basically, this action is described as cementation and/or densification.

7B.3.2 Injection Site—Location and Design for Procedure

Injection sites and depths of placement are dictated by soil borings or other means to determine depths and extent of fill as well as load carrying requirement of the structure. Grout injection sites are then configured in a pattern to provide proper coverage of area and competent placement of materials.

Injections are performed in a series of "lifts" to ensure that all applicable depths of the area receive the necessary quantities of grout. As a rule, the grouting proceeds from the bottom upward. Pumping is continued at each level until either of several predetermined events occur, such as: 1) placement of specific volume of grout, 2) pumping until some pressure is reached, or 3) communication of grout. In the event of communication (or "bleeding") problems, it often becomes necessary to stop pumping and relocate to another injection site until the grout in place reaches at least initial set. When the latter occurs, pumping can frequently be recontinued and carried to a satisfactory conclusion. In some severe instances, the use of a downhole packer becomes necessary in order to prevent the grout from communicating to the surface. Figure 7B.3.1 provides an example. This setup is particularly applicable when injection shafts are predrilled. Although there are many applications and varying techniques, Section 7B.3.7 discusses two case histories of deep grouting procedures.

Fishing is seldom required in this type of grouting, since the depths are relatively shallow [generally less than about 30 ft (9 m)] and are seldom in a critical location. This allows lost pipe to be merely abandoned after whatever pipe and fittings can be conveniently recovered. A new injection site is created and the process goes on. When this is not the case, pipe lost in the hole can be fished or recovered by using such tools as overshots, spears, or reverse spirals (to recover augers). This equipment is common to oil well drilling contractors and contractors engaged in projects such as dam, tunnel, or very deep (100 ft/30 m) grouting.

7B.3.3 Grout Composition

Specific grouts can involve such products as: polyurethanes, polyacrylamides, sodium silicates, cement, cement with admixes (i.e., fly ash or bentonite), etc. The specific product is selected based on costs, problems to be remedied, and placement conditions. In the cases focused on by this book, only compaction grouting (or variations thereof) and the use of cement-type grout will be considered. A broader discussion is provided in Section 6B of this book. By far and large, most of the "cement" grouts are simply a mixture of siliceous soil–cement and water. Strength and consistency of the grout is controlled by varying the solids–water and cement content. Frequently, this grout will be thicker (less water) and/or contain an increased cement content (higher strength) than conventional mudjack grout.

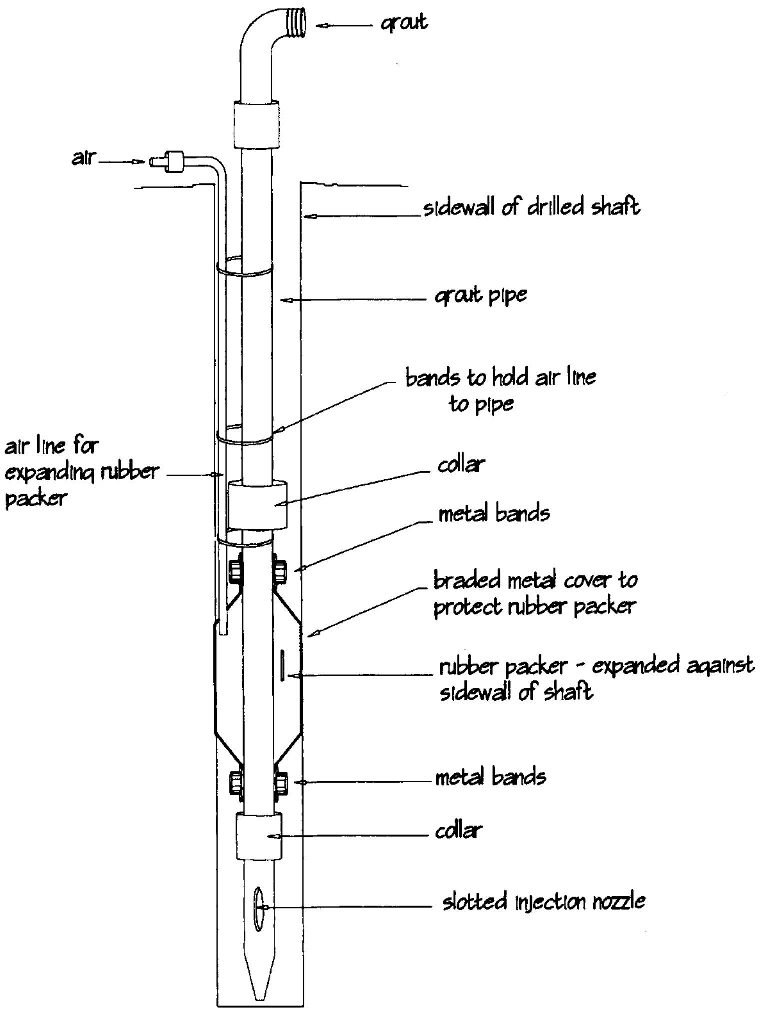
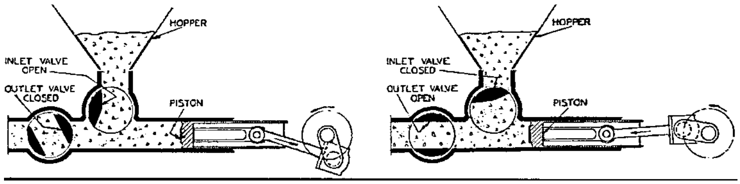


FIGURE 7B.3.1. Down hole packer for grouting.

7B.3.4 Mixing and Pumping Equipment

There exists a wide selection of grout mixers and pumps, the choice of which is largely dependent upon the specific project requirements. The operations considered herein are most often handled by the conventional mudjack equipment. Figure 7B.3.1 depicts an example of a common pumping and mixing unit. However, smaller, trailer-mounted concrete or mortar pumps (such as the Whitman), Colcrete, or Moyno pumps can also be quite handy. The latter equipment requires a separate mixing, which can be obtained from ready-mix trucks or conventional concrete (mortar) mixers. Refer to Figure 7B.3.2 for examples. Figure 7B.3.2a depicts a Moyno pump with auxiliary mixers. Fig. 7B.3.2b shows a Whitman concrete pump, which relies on a ready-mix truck to

(a)



(b)

FIGURE 7B.3.2. Pump and mixing equipment. (Chemgrout is a registered trademark of Robbins & Meyer, Inc.)

supply the material. The equipment for a particular job would be selected based on placement volume and pressure, as well as size and access to the work site. Tables 7B.3.1 and 7B.3.2 provide conversion of pressure units. Intermediate grouting as defined herein seldom requires actual placement pressures in excess of perhaps 50 to 100 psi (345 to 689 kPa). However, line friction must also be considered. The latter is influenced by the desired placement rate and size of conduit. Table 7B.3.3 presents data for estimating the pump power required for specific conditions of volume and pressure. The hydraulic horsepower divided by the pump efficiency (often 0.75) will give the corresponding brake horsepower. For example, if Table 7B.3.4 suggests the need for 70.2 HHP (30 yd^3/day at 250 psi), the brake horsepower required would be 94 (assuming 75% efficiency). If the efficiency factor is 60%, the brake horsepower required to deliver the desired HHP becomes 117. Table 4.3 offers *representative* values of friction pressure. These numbers can deviate from those field-recorded by a factor of two or more, depending largely on the specific grout composition. A more appropriate friction loss value could best be determined experimentally under actual field conditions. This can be accomplished by establishing the pump rate through an open-ended hose and recording the pressure at the pump discharge (or head).

TABLE 7B.3.1 Pressure Conversions, Pounds per Square Foot to Kilopascals

Pounds per square foot	psi	Kilopascals
1	144	0.0479
2	288	0.0958
3	432	0.1437
4	576	0.1916
5	720	0.2395
6	864	0.2874
7	1008	0.3353
8	1152	0.3832
9	1296	0.4311
10	1440	0.4788
25	3600	1.1971
50	7200	2.394
75	9800	3.5911
100	14,400	4.7880

7B.3.5 Placement Techniques

The foregoing and following paragraphs cover some of the questions concerning preplanning for grout placement. As noted, the grout injection pattern varies depending upon the particular problem to be resolved. About the only "uniform" criterion seems to be that to produce a grout curtain or consolidated mass requires a staggered injection grid consisting of at least three rows with perhaps a 4 ft (1.2 m) OC spacing. (Figure 4.3 also shows an injection pattern, although this was created to double as a pattern for subsequent mudjacking.)

TABLE 7B.3.2 Pressure Conversion, Pounds per Square Inch to Kilopascals

Pounds per square inch	psf	Water head, ft	Kilopascals
1	0.0069	2.31	6.895
2	0.0138	4.62	13.790
3	0.0207	6.93	20.685
4	0.0276	9.24	27.580
5	0.0345	11.54	34.475
6	0.0417	13.85	41.370
7	0.0486	16.16	48.265
8	0.0552	18.47	55.160
9	0.0625	20.78	62.055
10	0.069	23.09	68.950
25	0.174	57.72	172.375
50	0.345	115.45	344.750
75	0.52	173.17	517.125
100	0.69	230.9	689.500

TABLE 7B.3.3 Hydraulic (HHP) Horsepower Tables*

Cubic yard/day†	cfm	50 psi	100 psi	150 psi	200 psi	250 psi	500 psi
10	0.56	3.89	7.78	11.67	15.56	23.34	38.89
20	1.125	7.8	15.56	23.34	31.12	46.68	77.78
30	1.68	11.67	23.34	35.01	46.68	70.2	116.7
40	2.25	15.6	31.12	46.68	62.24	93.36	155.6
50	2.8	19.45	38.9	58.35	77.8	116.7	194.45
100	5.6	38.9	77.8	116.7	156	233	389
200	11.25	67.8	155.6	233.4	311	467	778
300	16.8	116.7	233.4	350	467	702	1167
500	28	194.5	389	584	778	1167	1945
1000	56	389	778	1167	1560		

*HHP = $Q \times P_D/7.2$, P_D in psi and Q in cfm. P_D = Total or delivered pressure = $P_C + P_C$ where P_f = Friction pressure and P_C = injection or compaction pressure.

†Day = 8 hr.

7B.3.6 Estimating Grout Volume Required

This is difficult to do because of the many unknowns. Consequently, most jobs are bid either on a per cubic foot or time and material basis. If some form of "guesstimate" is required, a couple of pointers might be useful:

1. Noncohesive soils. The *theoretical* void in a poorly graded granular material is 40%. Giving thought to reality, a workable estimate might be 20 to 30%. If the volume to be considered measures 100 ft (30 m) by 30 ft (9 m) by 20 ft (6 m) deep the soil volume would be estimated at 60,000 ft^3 × 0.25 or 15,000 ft^3 (424 m^3 or 556 yd^3). Any void created by erosion would be added to this.

TABLE 7B.3.4 Line Friction, ΔP, psi

	Feet of 2″ ID hose					
Q, cfm	50	100	150	200	300	cubic yard/day*
0.56			0.4	0.6	10	
1.125	0.5	0.6	1.0	20		
1.68	0.6	0.75	1.2	30		
2.25	0.65	0.85	1.30	40		
2.8	0.5	0.75	1.0	1.5	50	
5.6	0.5	1.0	1.5	2.0	3.0	100
11.23	1.0	2.0	3.0	4.0	6.0	
16.8	1.5	3.0	4.5	6.0	9.0	200
28	3.125	6.25	10	12.5	20	500
56	6.25	12.5	19	25	38	1000

This table assumes a soil–cement grout containing 50 gal H_2O per cubic yard. This table provides only a *broad* estimate, since the specific viscosity and density of the grout as well as appropriate friction factors are unknown. Where possible, the actual friction values should be determined experimentally. The desired, delivered pressure is added to the number indicated in the Table 7B.3.3 to arrive at the pump head pressure. This total becomes P_D (Table 7B.3.3), 1 cf = 7.48 gal.

2. Cohesive Soils. These contain little, if any, porosity under natural conditions. Voids occur as a result of organic decay, consolidation (compaction), or erosion. It is best to attempt to use all information available to "approximate" the void. The grout necessary to handle the problem will equal the volume of the void plus whatever compaction might be desired.

7B.3.6.1 Pricing Grout

The price for grouting is computed based on a per cubic foot in-place basis, which varies widely with the specific job condition. A *very* rough number might be on the order of $8.50 per cubic foot (1 $ft^3 = 0.028\ m^3$) of wet grout.

7B.3.7 Case Histories

Two jobs were selected to provide a fair overview of intermediate grouting. One project was performed in Dallas, Texas, and the other in Louisiana. Similar problems could be encountered anywhere.

7B.3.7.1 Dallas Hospital Streets

On January 28, 1986, a project was initiated by a Dallas hospital to grout an abandoned waste line that had experienced a washout to depth of approximately 12′ (3.6 m) below street level. The washout was created as a result of a 4″ (10 cm) water main break and had undermined the bearing soil of the concrete drive above. The remediation process selected was to fill in the void, restoring the bearing support to the road, followed by mudjacking the pavement.

Deep grouting involved the drilling and placement of 2″ (5 cm) OD grout pipes to a predetermined depth of approximately 9′ below finished grade of the road for placement of grout. Conventional mudjacking equipment was used to mix and pump the grout.

To assure competent placement, holes were drilled in the street surface along the sewer line at 6–8′ (1.8–2.4 m) centers (see Figure 7B.3.3b). The grout pipes were driven to the proper depth by pneumatic pavement breaker. Connections were fitted to the pipes following placement. A reinforced rubber hose was connected and then attached to a Model 50 Koerhing Mudjack. A soil–cement grout was pumped through the grout pipes into the subgrade. Grout placement was performed along the sewer lines area to refusal, which assured adequate filling of voids and restoration of bearing support. Upon refusal, it was possible to raise the settled areas of the road and parking garage ramp approximately 4″ (10 cm).

The area treated encompassed 10,000 ft^2 (930 m^2) and required the placement of approximately 38 yd^3 (29 m^3) of grout. Work of this magnitude required six days (with four-man crew), and was successfully completed without incidence. Refer to Figures 7B.3a and b for more detail.

7B.3.7.2 Louisiana Salt Mine—Hoisting House

In late 1988, a project was initiated by a salt mining company to level and stabilize the production hoist house at a Louisiana facility. The foundation was constructed with a perimeter beam 1′ thick × 4′ (0.3 to 1.2 m) in depth with a 1′ × 4′ wide footing poured integrally at its base (refer to Figure 7B.3.4). Twelve inch diameter (0.3 m) piers had been placed to a depth of 40′ (12 m) through the overburden soil to the lower salt dome. The interior floor was a 4″ (10 cm) thick steel-reinforced concrete "floating" slab.

Over the years, groundwater had penetrated the pier shafts, eroding the dome. The erosion of the salt in the pier shafts undermined the support to the production hoist house foundation, creating significant failure of the structure. Principally, the overburden material consisted of quicksand, with some loose gravel. It became apparent that the most practical approach to repair would be to strengthen the overburden soils through deep grouting, to create a suitable and competent subgrade. Due to space limitations and the design of the original beam, the proposed repair procedure required installation of spread footings with deep grout pipes cemented in the pour to facilitate raising and sustaining the foundation beam. Attempts to raise the perimeter were unsuccessful, due to the enormous structural load. Refer to Figure 7.B.4.4. Consequently, excavation along the entire outside foundation for removal of the 2′ (0.6 m) overburden on the strip footing (refer to Figure 7.B.4.4)

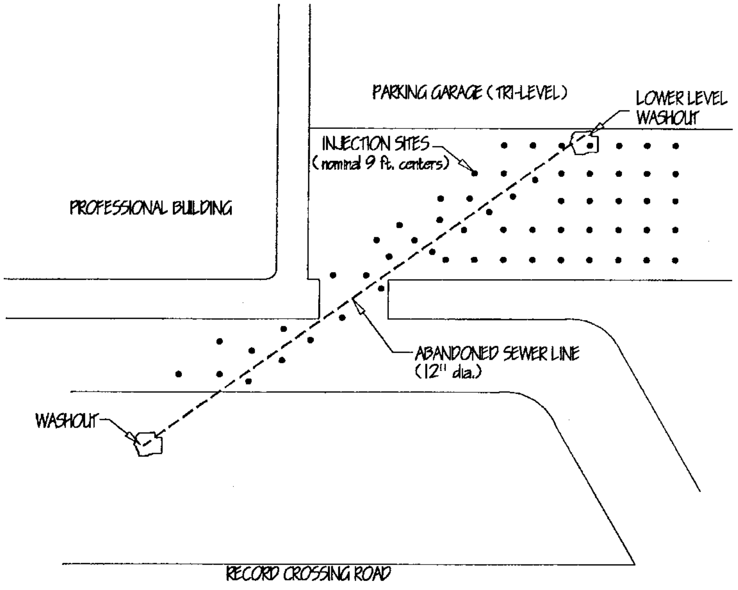


FIGURE 7B.3.3a. Dallas Hospital streets–injection pattern.

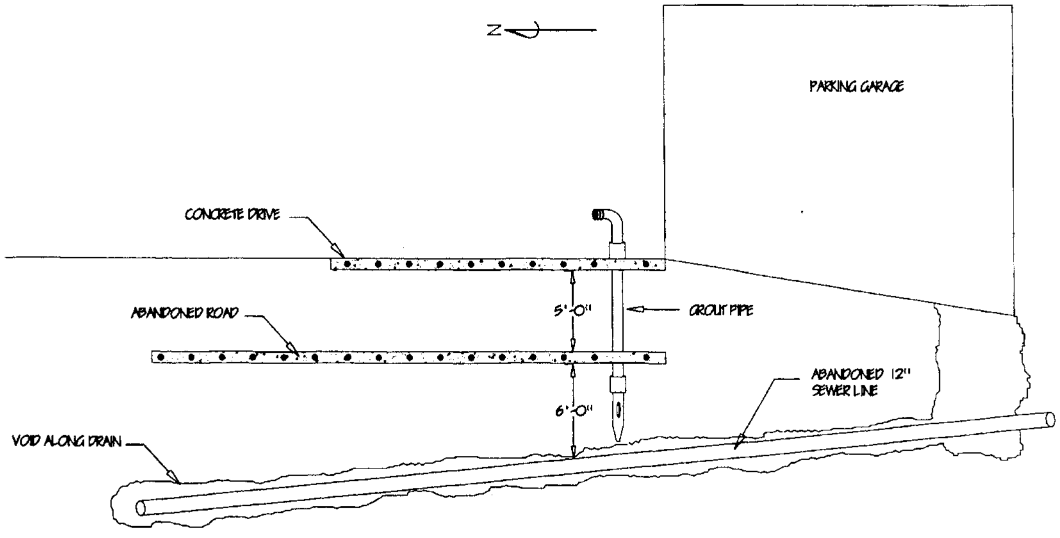


FIGURE 7B.3.3b. Hospital grout pipe placement.

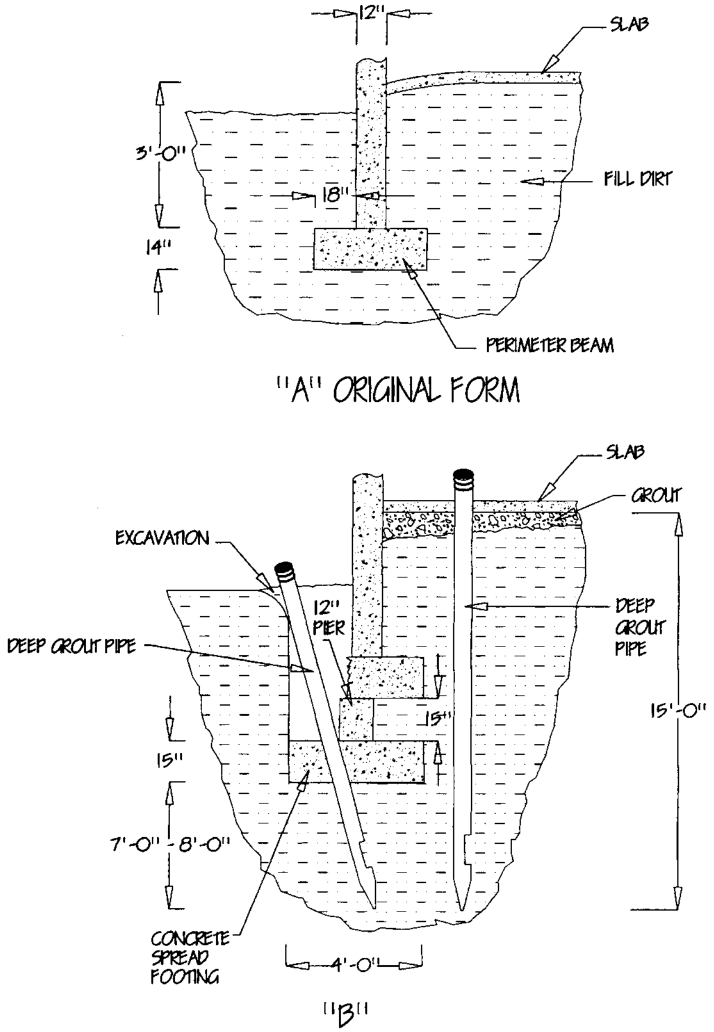


FIGURE 7B.3.4. Louisiana salt mine hoisting house project.

was performed. To further complicate matters, a beam similar to the one described above was found to transect the width of the interior slab.

7B.4 UNDERPINNING

Leveling efforts (underpinning and mudjacking) were attempted prior to the deep grouting of the footings to test both the strength of the subgrade and load of the structure. Leveling at this point was not possible. The beam could not be raised without risk of damage.

Grout pipes were placed as necessary throughout the interior slab to depths of approximately 15′ (4.5 m).

As mechanical raising of the beam was attempted, simultaneous injection of a bentonite slurry was injected on the back side of the beam, along with deep grouting to fill voids and densify the subgrade. The bentonite slurry reduced friction and the combination of these three procedures created conditions to allow raising of the beam footings. Grouting was performed to refusal with two "lifts" at 10′ and 5′ (3 and 1.5 m) below slab level.

Once the "deep" grouting was completed, mudjacking was performed on the interior slab to complete the leveling operations.

The production hoist house foundation area covered approximately 3500 ft^2 (325 m^2) and the required installation of 31 footings and the removal of over 130 yd^3 of soil around the perimeter. The deep grouting operation necessitated the placement of 234 yd^3 (99 m^3) of soil–cement grout. The pump house is in full operation as this goes to press.

7B.4.1 Introduction

A multitude of options have been tried over the years for underpinning foundations. These have included: a) conventional steel-reinforced concrete piers, b) steel-reinforced concrete spread footings, c) hydraulically driven steel minipiles, d) mechanically driven steel minipiles, e) screw anchors, f) hydraulically driven concrete cylinders, g) the ultraslim drilled concrete piers, and h) the hydropier, which is perhaps the least effective of the lot. Generally speaking, underpinning is relegated to the perimeter beam. On rare occasions, such as settled interior fireplaces, underpinning may be done inside the structure. When this does occur, it is quite expensive and requires breaking out sections of slab floors or removing wood flooring and subflooring in the case of pier-and-beam foundations. An interior underpin often costs as much as five times more than those placed at the perimeter. Tables 7B4.1 to 7B.4.4 in Section 7B.4.6 provide useful data on concrete and rebar.

7B.4.2 Conventional Drilled Shaft Piers

The conventional, drilled-shaft, steel reinforced, concrete piers pose many advantages. The optimum diameter for this pier is 12 in (30 cm) when used with lightly loaded structures (i.e., residential construction).[26,44] The piers are normally spaced on 7–9 ft (2.4–2.7 m) centers. The shafts normally extend to a depth of: a) adequate bearing capacity, b) undisturbed native soil, c) below the local SAZ (soil active zone), d) rock contact, or e) acceptable and specific site conditions. Frequently, the accepted depth is 9–15 ft (2.7–4.5) below surface. The shafts can be straight or belled. Belling may be required to control upheaval or, in some cases, to enhance the bearing capacity of the pier in substandard soil. The reinforcing steel is at least 2 #3s but can be as many as 4 #4s or 4 #5s. The piers are poured monolithically with a haunch usually 30″ × 30″ × 12″ (0.9 × 0.9 × 0.3 m), which is also steel-reinforced and integrally tied into the pier shaft.[15–17] After adequate concrete curing time, the haunch is used as a base from which to raise the perimeter beam. Once the beam is mechanically jacked to proper grade, a form is set and a pier cap poured. (Although the pier is classically in compression, a rebar is generally centered in the cap, mostly as a control for any lateral movement.) The concrete is poured into contact with the foundation beam (refer to Figure 7B.4.1). This serves two important purposes: first, the concrete cap is in intimate contact with entire irregular beam surface and second, the use of shim material is avoided. Unlike to shim materials, the concrete is not subject to deterioration.

Figure 7B.4.1 presents two options that represent acceptable pier designs. The designs are verified by a simple study of pier moments and haunch bearing. The oversimplified mathematical analysis shows the piers to be safe for the representative conditions. It is always wise to subject any underpinning approach to the same scrutiny. Along these lines, refer also to Figure 7B.12. Table 7B.4.5 in Section 7B.4.6 provides data that might be useful for this purpose. The following analysis

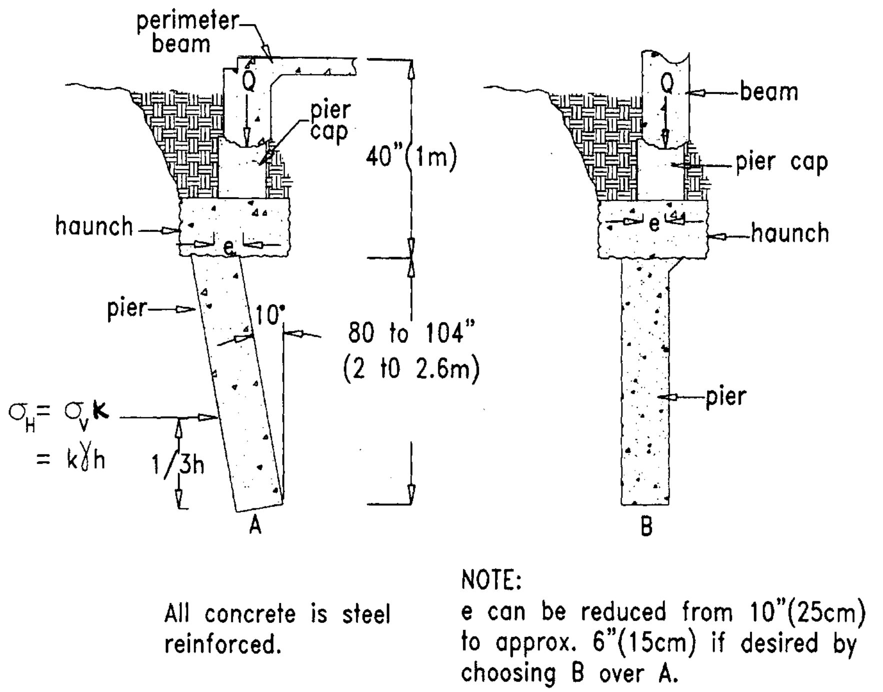


FIGURE 7B.4.1. Two acceptable drilled piers.

does not address the structural competency of the pier itself, as this has already been established. The principal concern is to evaluate the soils capacity to resist the loads.

For study purposes, the structural load was assumed to be 500 lb/l ft of beam, the soil unit weight was assumed to be 110 lb/ft^3 (17 kN/m^3) and the soils' unconfined compression strength was assumed to be 1500 lb/ft^2 (7320 kg/m^2).

Bearing haunch only

$$Q_L = 500 \text{ psf} \times 8 \text{ ft} = 4000 \text{ lb (4 k)}$$

$$Q_R = 1500 \text{ lb/ft}^2 \times 2.5 \text{ ft} \times 2.5 \text{ ft} = 9375 \text{ lb}$$

Safety factor (SF) = 9375/4000 = 2.3

Where Q_L = wt/ft × ft = load

and $Q_R = q \times a$ = resistance

For moments: piers only

A

Q_L = 500 plf × 8 ft
= 4000 lb = 4 k

$M_L = e \times Q_L = 7'' \times 4000$ lb
= 28,000 in-lb = 2.3 ft-k

B

Q_L = 4 k

M_L = 10/12 ft × 4k = 0.833 ft × 4 k
= 3.3 ft-k

(e can be reduced by shaving inside of pier shaft.)
Q_L and M_L are results of load
$\varnothing = 35\%$

For soil resistance to lateral movement

10 ft pier, 12″ diameter, M_R = moment resistance
$\sigma_H = K_a \gamma H$, where K_a = coefficient of passive stress.[65]
$\sigma_H = 10 \text{ ft} \times 110 \text{ lb-ft}^2 \times 0.33 = 363 \text{ lb/ft}^2$, assume $K = 1.0$[16,17]
$P_a = \frac{1}{2}(363)(10) = 1815 \text{ lb/ft}^2$
$M_R = (1)(1815)\ 6.7 \text{ ft} = 12{,}160 \text{ ft} \times \text{lb}$
$= 12.16 \text{ ft} \times \text{k}$

The safety factors are

A	B
$SF_A = 12.16/2.3 = 5.3$	$SF_B = 12.16/3.3 = 3.7$

If the pier depth is increased to 12 ft, the M_R for A and B becomes 14.5 and the factors of safety increase to 6.3 and 4.4, respectively.

Obviously, the safety factors are sufficient for the underpinning being considered. The safety factor for the straight shaft can be improved to equal or exceed that shown by the raked pier by drilling the pier further toward the center line of the beam. In fact, e can be handily reduced to less than 6″ (15 cm).

The presence of the haunch increases the movement resistance of the soil as the presence of the pier increases the load capacity. Once the soil bearing is restored to the foundation beam between pier locations, the load on each pier is *decreased* by a factor of about 8 and the resultant movement on the pier is reduced by the same amount. The raked pier (A) can be drilled with a full auger bit, whereas the straight shaft (B) cannot. This makes the (A) shaft a little quicker to drill.

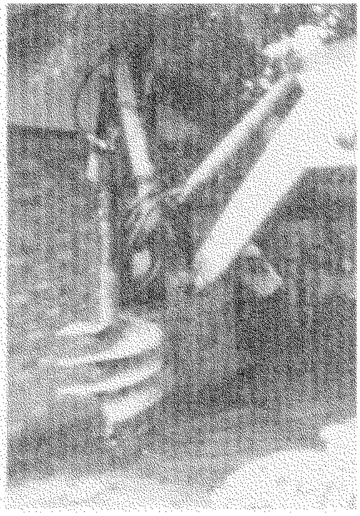
7B.4.2.1 Drilling Pier Shafts

Figure 7B.4.2a through c depicts equipment used to drill piers. The truck-mounted unit represents the quickest and least expensive means for drilling but requires access and head room. (The expense advantage becomes even more pronounced as the pier depth and diameter increase.) This equipment can drill rock. The truck-mounted drill is also used to provide shafts for deep grouting, sometimes to a depth of 100 ft (30 m). The tractor rig is effective when access is limited, shaft diameters are 24 in (0.6 m) or less, and depths are less than about 30 ft (9.1 m). This equipment can cut soft rock and bell a 24 inch (0.6 m) shaft to 42 in (1.4 m). The latest equipment is the limited access rig, which is used in those cases where access is critically limited.

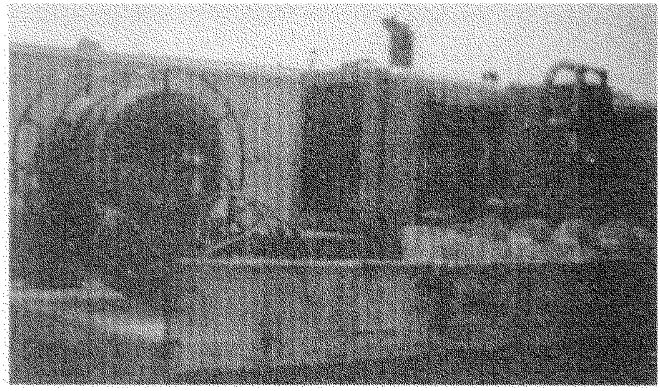
This equipment can drill with 7 ft (2.1 m) head clearance and no more than 4 to 5 ft (1.2 to 1.5 m) lateral or surrounding access. Drill stems and extensions are generally 4 or 5 ft (1.2–1.5 m) in length. This necessitates considerable hand work running or pulling the bit. The rig pictured is capable of drilling a 12 in (0.3 m) shaft to a depth of 20 ft (6 m) and providing a 24 in (0.6 m) bell. The unit shown is capable of delivering 1600 ft-lb (2170 Nm) torque, 5000 lb (2300 kg) lift and crowd, and about 200 max. rpms. The deterrents are the time required to drill (and bell) a pier and the inherent inability to penetrate rock to any degree. The machine can, however, drill smaller-diameter, straight shafts to relatively shallow depths at reasonable costs. For example, a 12 in diameter (0.3 cm) pier can be drilled to 12 ft (3.6 m) for a cost roughly $130 more than that for the truck rig and $90 more than that for the tractor-mounted machine. The cost to drill a limited access pier is slightly higher than that associated with the excavation for a spread footing. The same pier can be drilled with either of the other machines at a cost below that required to excavate the spread footing.

Unless the truck rig remains on paved surfaces, considerable yard and landscaping damage could result. The tractor rig also tends to create similar (though smaller) surface disturbances. This is especially true where the yards are less than dry. Figure 7B.4.3 illustrates the drilling sequence

(a)



(b)

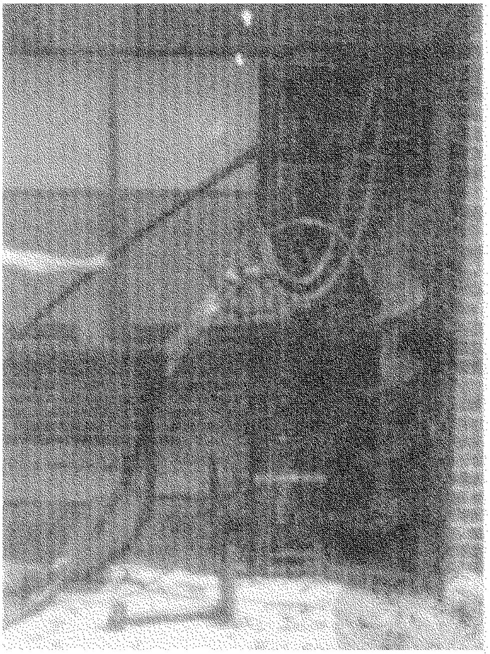


(c)

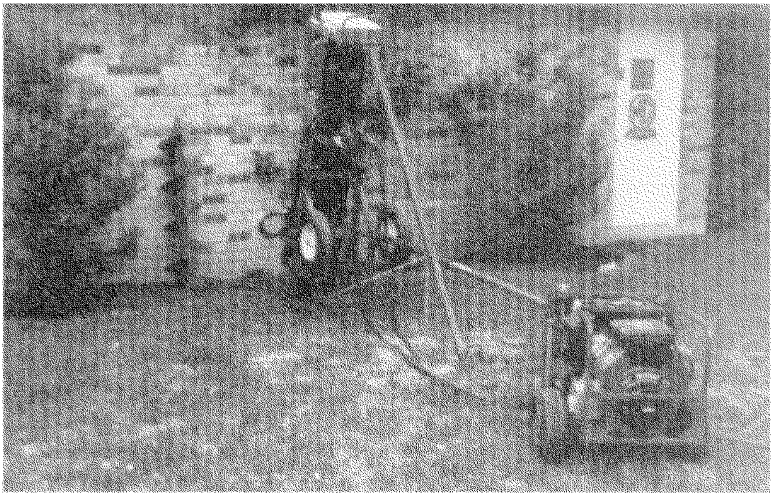
FIGURE 7B.4.2. Pier drilling equipment. (a) Truck-mounted drill; (b) tractor drill. (c1) limited-access drill.

for pier shaft preparation. Figure 7B.4.4 depicts the pier construction, and Figure 7B.4.5 pictures the haunch and final raising. Drilled shafts must be poured as quickly after drilling as possible. Every attempt possible should be made to pour concrete into shafts on same day they are drilled.

In some parts of the United States (Florida for example), the pier is poured by pumping a concrete grout (cement–sand–water) through the drill stem as the bit is pulled. This could be useful if sloughing or water intrusion is to be encountered. Rebar is placed into the shaft after the bit is removed. The concrete mix consists of approximately 1034 lbs (469 kg) cement, 55 gal (208 L) water, and 20 ft^3 sand per cubic yard of concrete. These piers, 12″ dia (3 m) to 12 ft (3.6 m), cost the customer about \$1000 (U.S. 1998 dollars).

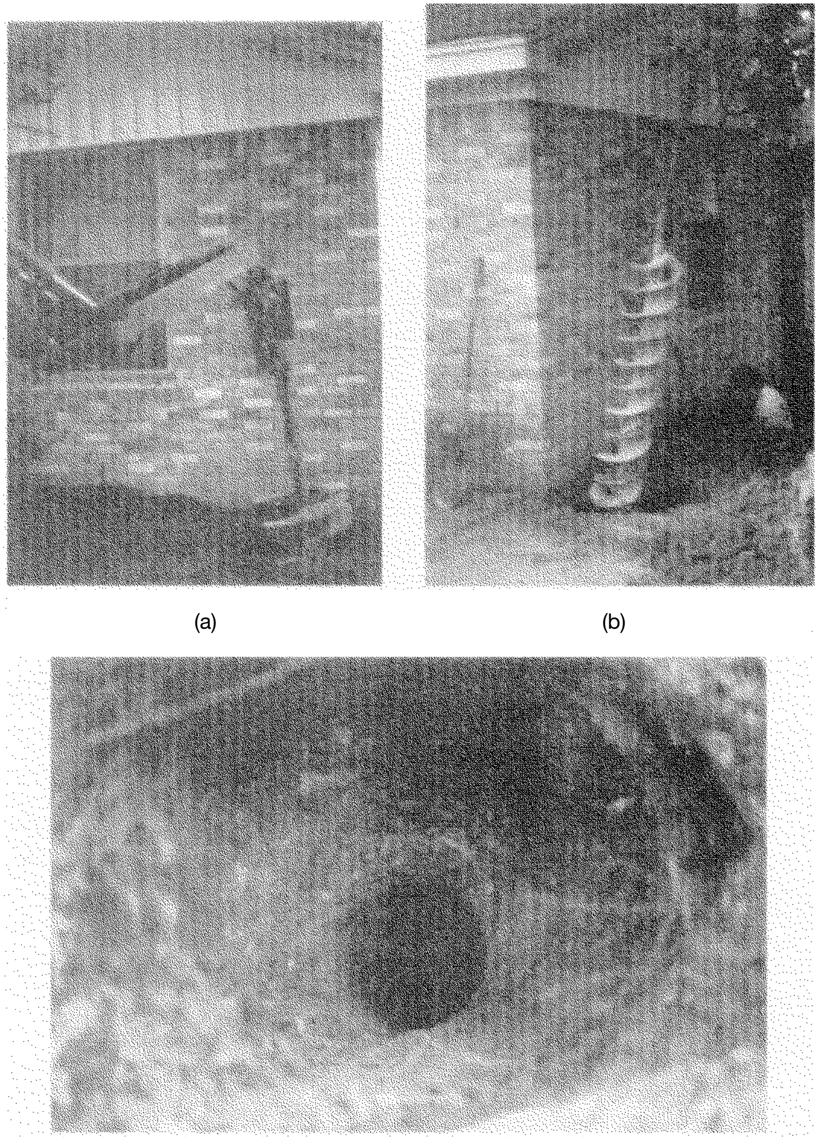


(c)



(d)

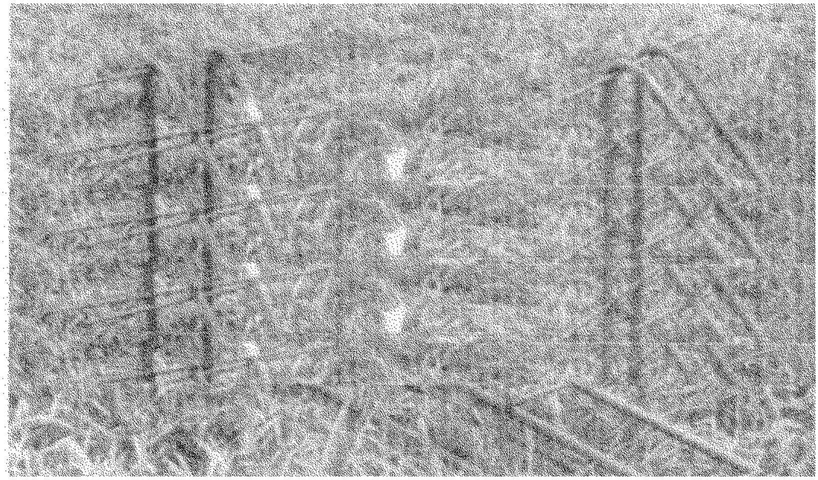
FIGURE 7B.4.2. (*continued*). Pier drilling equipment. (c2) limited-access drill. (d) Lighter, limited access equipment is also available. One such model, the Big Beaver, weighs only 500 lb (230 kg), has a lifting capacity of 1650 lb (540 N × m) torque. The company advertises a maximum bit size of 18 in (46 cm) with depth capability to approximately 20 ft. The power unit is located at lower right. This equipment is basically a substantially downsized copy of the limited-access equipment depicted in (c1) and (c2). The significant difference is the Kelly drive system. The Big Beaver utilizes a screw drive, and the larger limited-access drill uses a chain drive.



(a) (b)

(c)

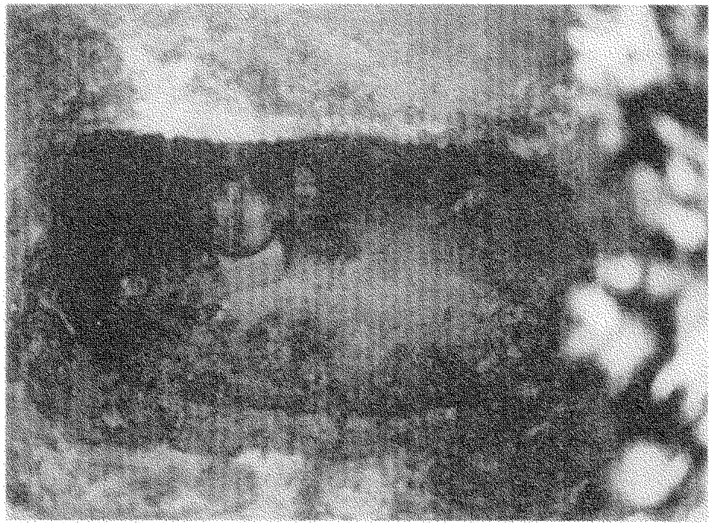
FIGURE 7B.4.3. Drilling sequence used for preparation of concrete piers for underpinning. (a) Drilling the pilot hole for haunch with 24 in bit. (b) Drilling the 12 in diameter shaft to 15 ft depth. The man at right is enlarging and shaping the haunch. (c) Haunch and pier ready for reinforcing steel and concrete.



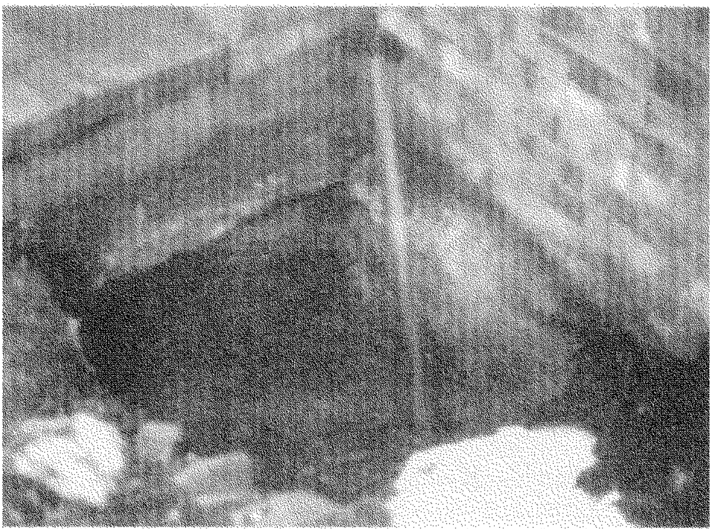
(a)

(b)

FIGURE 7B.4.4. Constructing the concrete pier. (a) Rebar caged, ready to place into pier hole (commercial application); (b) pouring the concrete.



(a)



(b)

FIGURE 7B.4.5. Underpinning and raising the foundation. (a) Haunch cured and jack in place for raising the perimeter beam. (b) The final step—the beam has been raised the pier cap poured. The steel pipe beside the pier cap is intended only to provide temporary support to the beam until the concrete pier cap cures.

7B.4.2.2 Designing the Pier

Generally, the piers or pilings depend upon end bearing and skin friction (except in high-clay soils) for their support capacity. In expansive soils, the design contribution for skin friction is disregarded for the top 7 ft (2.1 m) or so of the shaft. Piers or pilings are normally extended through the marginal soils to either rock or other competent bearing strata. This obviously enhances and satisfies the support requirement. The use of piers (or pilings), therefore, is generally restricted to instances where adequate bearing materials can be found at reasonably shallow depths. Generally speaking, for residential repairs, a competent stratum depth below about 20 ft (6.0 m) resists the use of the pier (or piling) technique. This depth concern might be lessened, however, by such actions as: enlarging the haunch, belling the pier shaft, compaction grouting or some combination of these. Greenfield and Shen[44] suggest an optimum pier diameter of 12 in (0.3 m). The theory is to minimize pier heave, provide adequate bearing, and, at the same time, facilitate proper steel reinforcing. [It is impractical, if not impossible, to utilize caged rebar in a pier diameter of less than about 10 in (25 cm).] The pier can be belled whenever the soil bearing capacity is low or questionable. The same reference also notes that pier spacing should be the maximum consistent with beam design, load requirements, and site characteristics. The logic herein is to provide maximum safe load (surcharge) on the pier to counter expansive soil heave. (Data published by Budge et al. suggest that increasing the surcharge load by a factor of two reduces swell potential in pure montmorillonite by a factor of 4.28.)[22] While Greenfield and Shen principally address original construction, the same design concerns would be applicable to remedial procedures.

7B.4.2.3 Raising the Perimeter Beam

Figure 7B.4.5b illustrates a pier cap in place. The beam is raised by the same technique as that described in Section 7B.1. The jacks used are generally 25 to 35 ton (22,700 to 31,780 kg) Norton or Simplex journal jacks or hydraulic jacks of similar capacity. Aluminum body 25 ton journal jacks cost slightly less than $1000 and weigh about 40 lb (18 kg). The 25 ton hydraulic jacks cost about one fourth and weigh about half as much. A variation is to use hydraulic rams with an auxiliary power source (such as the unit shown in Figure 7B.4.c1). By use of a manifold and selective control valves, multiple rams can be used simultaneously to literally "float" the structure to the desired grade. (The jacks in other procedures are individually and manually operated.) Occasionally, the need arises for greater jack capacity. When this occurs, the first approach is to work several jack locations simultaneously. (Essentially, the pier locations are spaced about 8 ft (2.4 m) apart.) If this doesn't work, 50 ton jacks (or larger) can be attempted. Special attention must be given to the configuration and load capacity of the concrete beam. It is important that the beam not be cracked.

Once the beam is satisfactorily raised, a steel pipe section is wedged in the space between the base and the haunch (to the side of the jack). The jack is then removed and a sonotube set in place to act as a form for the concrete pier cap. The pier cap is poured and the excavation back filled.

7B.4.2.4 Cost of Pier Construction

The material used to prepare the pier system shown is Figure 7B.4.6a would be:

Concrete:	All Concrete is 3,000 psi	
Pier:	$0.785\ ft^2 \times 10\ ft$	$= 7.85\ ft^3$
Haunch:	$2\ ft \times 2\ ft \times 1\ ft$	$= 4\ ft^3$
Pier Cap:	$0.785\ ft^2 \times 1.25\ ft\ (15'')$	$= \underline{0.98\ ft^3}$
		$12.8\ ft^3$ ($0.475\ yd^3$ or $1.19\ m^3$)

Steel:	All Bars are #3s (⅜ in or 0.9 cm)	
Pier:	$4 \times 10\ ft + 4 \times 2'$ (ties)	= 48 ft
Haunch:	12×1.5 ft	= 18
Pier Cap:	1×1.5 ft	= $\underline{1.5}$
		67.5 ft (20 m)

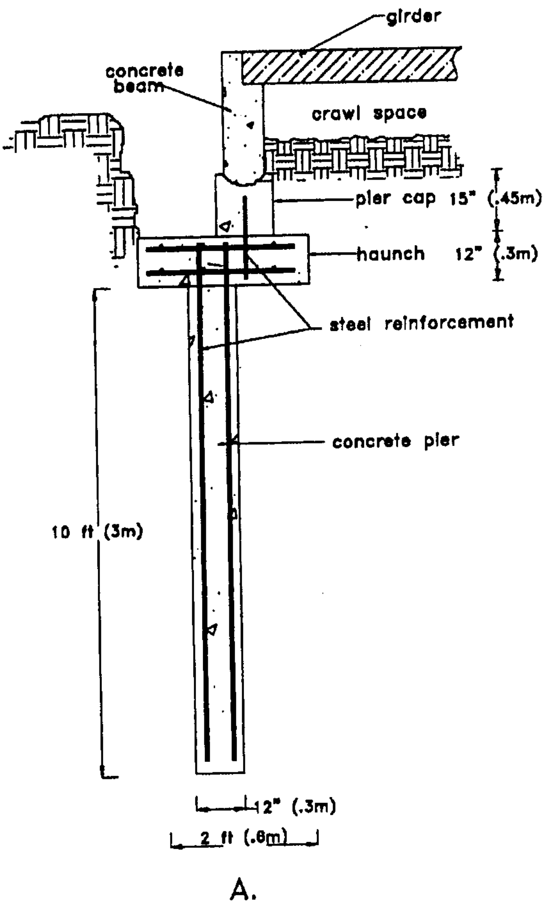


FIGURE 7B.4.6. Sustaining the perimeter beam.

The cost for the installation of the 12″ diameter (0.3 m) piers, spaced as shown in Figure 7B.4.6b, could be estimated as follows. Assuming access and no costly delays or interference cause by others, the typical pier should cost about \$300.00 to \$500.00 (U.S. 1998 dollars). This also assumes that the number exceeds the company minimum (which is often 7 to 10 units). The cost for less than 3 or 4 piers escalates by a factor of 1.5 to 2.0. (All prices are based on unskilled labor costs of \$7.00 per hour and concrete at \$60/yd^3.)

7B.4.3 Spread Footings

Typically, the spread footings consist of: 1) steel-reinforced footings of sufficient size to adequately distribute the beam load and poured to a depth relatively independent of seasonal soil moisture variations, and 2) a steel-reinforced pier cap tied to the footing with steel and poured to the bottom of the foundation beam (Figure 7B.4.7). Design and placement of these spread foot-

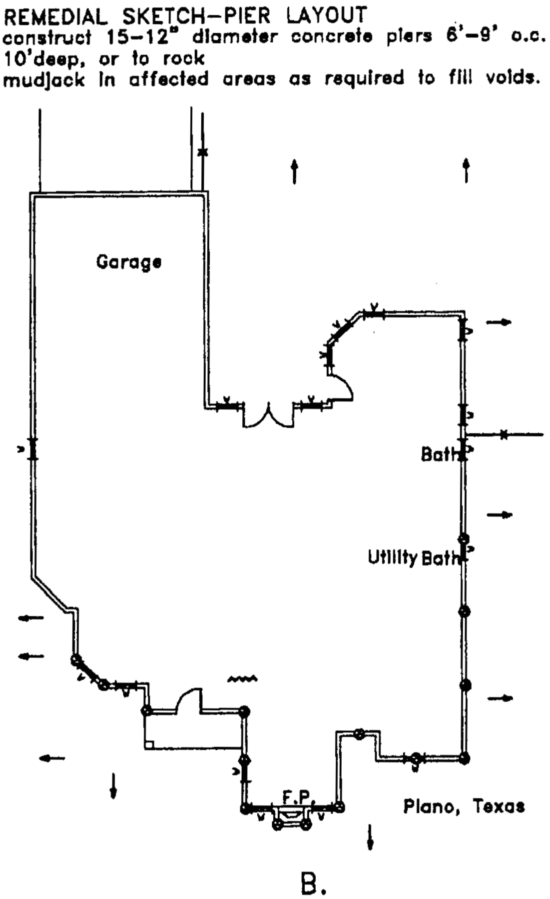


FIGURE 7B.4.6. (*continued*). Raising the perimeter beam.

ings is critical if future beam movement is to be averted. Nominally, the footings are placed on 8 to 9 ft (2.4 to 2.7 m) centers. The footing design must consider the possible future problems of both settlement and upheaval and should be of sufficient area to develop adequate bearing by the soil. Often, the pad is 3′ × 3′ × 1′ thick (0.9 m × 0.9 m × 0.3 m), located at a depth of 30 in (0.75 m) below the perimeter beam. The pier cap should be of sufficient diameter or size to carry the foundation load. It also should be poured to intimate contact with the irregular configuration of the undersurface of the beam. Precast masonry, steel, or wood shims should not be considered as pier cap material. The photographs in Figure 7B.4.8 depict actual field development of a typical spread footing. Figure 7B4.8a shows a spread footing base pad poured in place. Figure 7B.4.8b illustrates the pier poured. The jack used to raise the beam is still in place to the right of the pier and the steel pipe used to temporarily support the beam until the concrete pier cures is evident to the left of the pier.

The principle of the footing design is to distribute the foundation load over an extended area at a stable depth and thus provide increased support capacity on even substandard bearing soil.

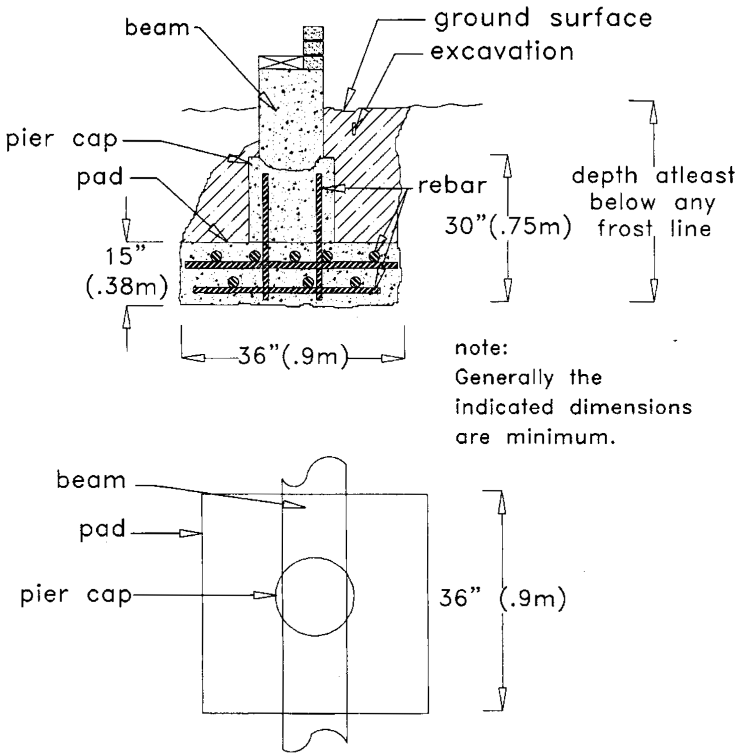
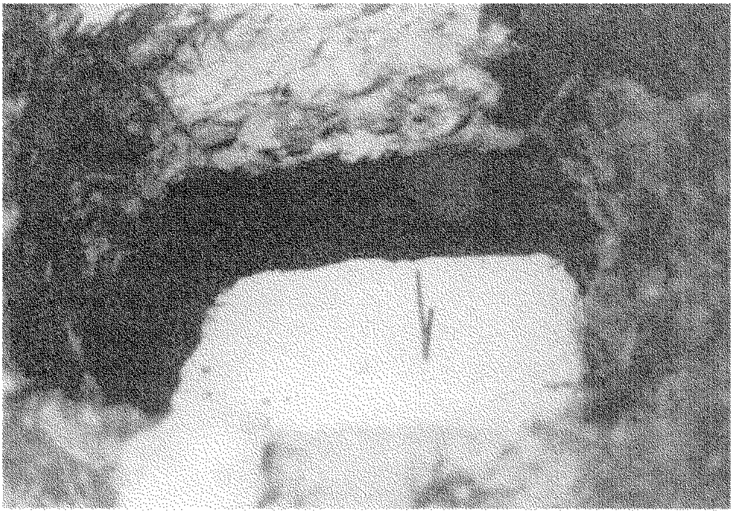


FIGURE 7B.4.7. Spreadfooting.

The typical design represented by Figure 7B.4.7 provides a bearing area of 9 ft^2 (0.8 m^2). Effectively, a load of 300 lb/ft^2 (1465 kg/m^2) applied over 1 ft^2 (0.09 m^2) would require a soil bearing strength of 300 lb/ft^2 (1465 kg/m^2). The same load distributed over 9 ft^2 would require a soil bearing strength of only 33 lb/ft^2 (161 kg/m^2). Expressed another way, a 9 ft^2 spread footing on a soil with an unconfined compressive strength (q_u) of 1500 lb/ft^2 (7320 kg/m^2) would provide a load resistance of 13,500 lb (6136 kg) ($Q_r = q_u A$). This capacity would exceed the structural loads imposed by residential construction by a wide margin. Generally, the diameter of the pier cap is greater than, or at least equal to, the width of the existing beam. The form for the pier cap must extend outside the beam in order to permit placement of concrete. Since the pier is essentially in compression, utilizing the principal strength of concrete, its design features are not as critical as those for the footings.

As previously stated, the spread footing should be located at a depth sufficient to be relatively independent of soil moisture variations due to climate conditions (SAZ). In London, this depth is reportedly in the range of 3 to 3.5 ft (0.9 to 1.1 m).[54] In the United States, this depth is reportedly in the range of 2 to 3.5 ft (0.6 to 1.1 m).[102] In Australia, the depth is considered to be less than about 4 ft (1.25 m).[52,53] In Canada, this depth has been reported to be as shallow as 1 ft (0.3 m).[97] Along this line of thought, Fua Chen presented a paper that questions the reliability of theoretical approaches to predict heave.[27] The heave prediction methods are based on an assumed depth of wetting, which varies considerably among investigators. Chen also suggests that heave predictions are generally much greater than those actually measured in the field. Does this mean that the seasonal depths of soil moisture change are, in fact, considerably less than values normally assumed?



(a)

(b)

FIGURE 7B.4.8. (a) Concrete pad is poured in place (note rebar in place). (b) The concrete pier cap has been poured on the pad to support the foundation beam. The jack used to raise the beam is still in place, and the steel pipe used as a temporary support is in place.

Nonetheless, proper soil moisture maintenance would ensure the stability of the footing with minimal concern for the effective "active depth" or depth of ambient soil moisture variation. One concern that might arise would be the effectiveness of the pad to support the structural load. Generally, this is a nonissue because of the enormous load capacity of the pad (9 ft^2 or 0.81 m^2). However, it can be noted that if the pad cracks under loading, it has failed! If the pad sinks, the bearing soil has failed.

7B.4.3.1 Construction of Spreadfooting

The procedure for construction and implementation of spreadfooting is about the same as that outlined in Section 7B.4.2 for a drilled pier. First the excavation is dug as shown in Figures 7B.4.7 and 7B.4.8. Soil Sta is poured into the excavation as a safeguard against future upheaval should some source provide water to that depth. Soil Sta will be discussed in some detail in Section 7B.5. Next, the spreadfooting is poured and allowed to cure. Generally, the footing thickness is in the range of 12 to 15 in (0.3 to 0.38 m). This is much thicker than the structural demand would require. However, from a practical standpoint, the design of the spreadfooting is dependent on choosing a depth reasonably free of seasonal moisture variations (30 in or 0.75 m for Dallas, Texas conditions). The 9 ft^2 (0.09 m^2) bearing area allows for a safety factor and permits the same design to be used in areas with soils of lower safe bearing strengths. The base of the spreadfooting shown in Figure 7B.4.7 is about 30 in (0.75 m) below the base of the beam and generally over 40 in (1.0 m) below grade, depending upon the depth of the beam. Next, the clearance required for the jacks is about 15 in (0.38 m). This space permits the use of a wood block on the jack head to help distribute the load and prevent the jack from slipping off the concrete beam. Hence, the thickness of the spreadfooting is determined by 30″ (0.75 m) minus 15″ (0.38 m) or 15″ (0.38 m). The completion of the installation is as described for the drilled pier.

7B.4.3.2 Cost

The material required for the spreadfooting in Figure 7B.4.7 would be:

Concrete: (3000 psi)

Spreadfooting:	2½ ft × 2½ ft × 1.25 ft = 7.8 ft^3
Pier cap:	0.785 ft^3 × 1.25 ft (15″) = 0.98 ft^3
	8.78 ft^3 (0.3 yd^3 or 0.8 m^3)

Steel (No. 3s):

Spreadfooting:	6 × 2 ft = 12 ft
Pier cap:	1 × 1.5 = 1.5 ft
	13.5 ft (4 m)

The installed cost for a spreadfooting would be something like \$300.00 to \$525.00, based on the same assumptions as those expressed for the drilled pier.

7B.4.4 Alternatives

Over the past few years, several alternative underpinning methods have been introduced. The following paragraphs will present a brief glance at many of these along with a critical review of their individual strengths and weaknesses.

The practice of underpinning without proper mudjacking has become a matter of litigation. It seems that a home owner filed suit against a repair company on the general grounds that the "addition of pilings changed foundation from soil supported concrete slab to slab supported by deep perimeter pilings." (Certainly, without proper mudjacking to restore the bearing, the plaintiff would be correct in these allegations). The Texas Court agreed with the plaintiff.

7B.4.4.1 Steel Minipiles

The steel minipiles that have been installed are eccentric (majority), concentric, concrete filled, equipped with helix(s), etc.[15–17,21,23,39,57,77,80,86,91] As a rule, the minipiles are located on 3–6 ft (0.9–1.8m) centers. By far and large, the minipiles, regardless of design, have not appeared to enjoy any real success when used in areas with expansive soils.[16,23,39,44,80,86] (Figure 7B.4.9 shows photographs of typical failures in minipiles.) First, the eccentric minipiles are perilously subject to failure due to bending moment and/or lateral stress.[15–17,23] Second, the weight of the structure serves as the reaction block to *hydraulically* drive the pier. Once the resistance on the pier *exactly* equals the weight of the structure, upward movement of the foundation occurs.[16,23] This relates to 1.0 margin of safety. It is true that piers can be "superloaded" by selective driving; however, this can subject the beam to excessive shear stress and possible ultimate failure. [The safety factor problem can be alleviated by mechanically driving the pipe (impact or torque); however, the other noted deficiencies remain serious concerns.]

Bolting the lift bracket to the perimeter beam often creates many structural concerns. Figure 7B.4.10 is a photograph that shows both severe damage to the perimeter beam and failure in the attachment of the lift bracket to perimeter beam. Another problem is the screw anchors, which are literally screwed into place. This action disturbs the soil traversed by the screw. The disturbed soil, in effect, acts as a "wick," which tends to pull moisture down to the lowest helix. This has caused serious foundation heave in both new construction and remedial applications.[39]* Refer also to A. Ghaly and A. Hanna "Uplift Behavior of Screw Anchors," I and II, *Journal of Geotechnical Engineering*, May, 1991. Also, minipiles can be mechanically driven by pneumatic hammering at the driving end inside the pile. A cushion of sand prevents or minimizes damage to the end of the pile. This approach allows for both better alignment and a margin of safety. These piles still suffer the other problems inherent to steel minipiles.

There is a type of sixth minipile that is driven *concentric* to the load (Information from Freeman Piering Systems, St. Louis, Missouri).[16] These afford better alignment but also suffer the same inherent defects common to steel minipiles. They introduce two other serious concerns—the persistent requirement for shoring and an inflated cost due to the additional excavation. See Figure 7B.4.11. These piers often cost in excess of $1000.00 each.

7B.4.4.1.1 Analysis of Eccentric Pile Driving The mechanics of pile driving is an issue of great concern, particularly in light of behavior patterns disclosed in Figures 7B.4.9 and 7B.4.10. Figure 7B.4.12 depicts a simplified analysis of pipe behavior during eccentric driving.

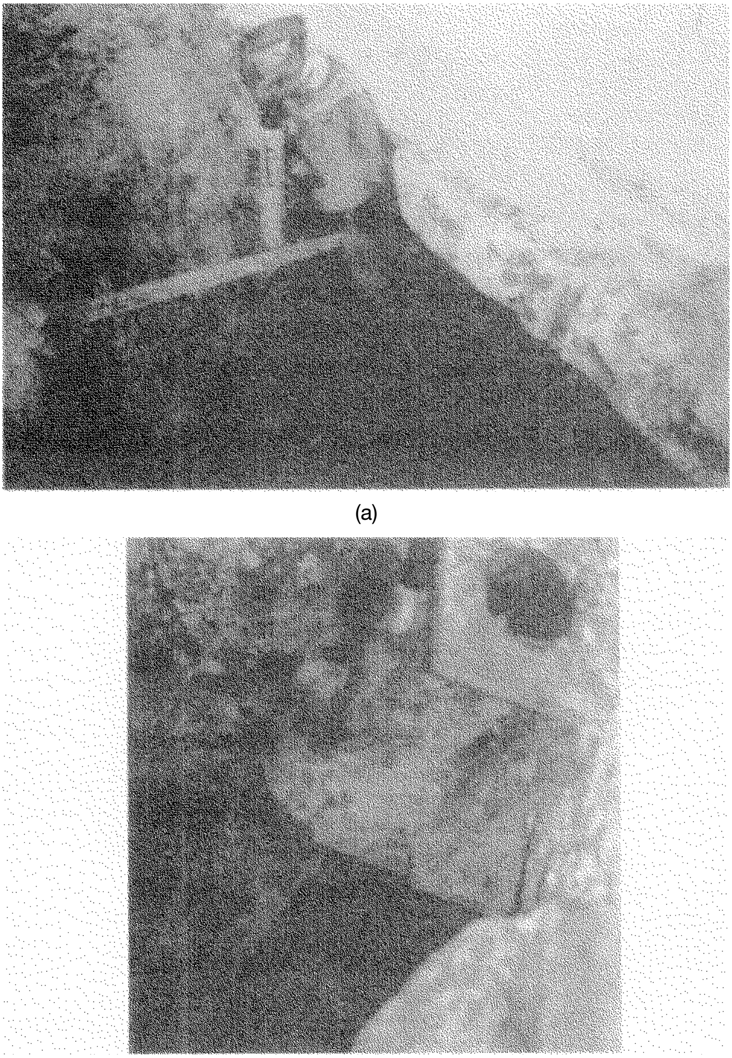
The first pile section would be forced outward (away from load Q_L) due to greater moment (2475 ft-lb vs 84 ft-lb resistance, Q_F). The second joint of pipe would then push the first joint at an outward angle with a vectored vertical force (resultant). The mathematical analysis suddenly becomes more complex. The type of connection would further influence this motion; i.e., slip fit coupling would permit a hinge effect, whereas a solid (welded) connection would simulate a single rigid pipe. For the former, the pipe configuration will be quite disjointed in appearance with each joint assuming a different direction (refer to Figure 7B.4.12b). The welded joint pipe behaving as a single long pile would assume a profile similar to an elongated "S" (refer to Figure 7B.4.12c). (Both configurations assume a homogeneous soil with no obstructions.) As additional joints are driven (pile lengthens), the force, Q_L, becomes less significant and the driving force, Q_J, and the soil's resistance, Q_R, become controlling factors. Accordingly, the complexities of the process become formidable issues. The degree of deflection is generally indeterminate. This is complicated even further by the pier's resistance to bending. However, it is a certainty that the piles will deviate from vertical to the extent that all vertical support capacity is threatened if not lost. The final failure may be delayed for some period of time (refer to Figure 7B.4.9).

The author gives special thanks to Dr. Richard Stephenson, Department of Civil Engineering, University of Missouri–Rolla, and to Dr. S. N. Endley and Dr. K. Mohan Vennalaganti, PSI Inc., Houston, for their valuable input on pile behavior under eccentric loading.

7B.4.4.1.2 Design Concerns for Minipiles. As a matter of interest, the following facts were taken from the authoritative British publication, *Building Research Establishment Digest, 313,* "Mini-piling for low-rise buildings"[23]

1. Minipiles are ineffective where: (a) lateral soil movements can be reasonably anticipated, i.e., expansive soils, sloping sites, soil consolidation, collapsing soils (for more about lateral stress, refer to the paper by Edil and Alanozy, "Lateral Swelling Pressures," Seventh Interna-

*Dr. Stephenson, C.E., Professor, University of Missouri–Rolla questions the "wicking." However, the problems associated with anchor heave are well documented in the Dallas–Fort Worth, Texas, area.



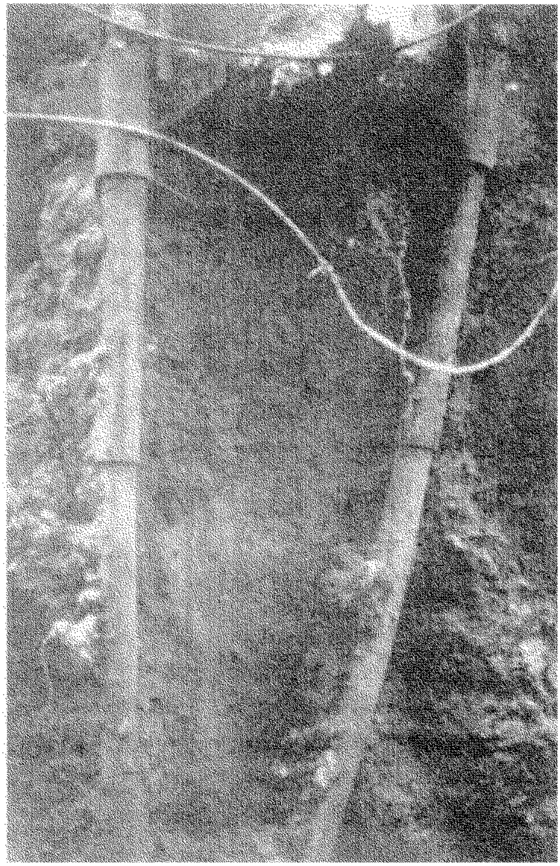
(a)

(b)

FIGURE 7B.4.9. (a) and (b), minipile failure.

tional Conference on Expansive Soils, ASCE, Dallas, 1992); (b) the appropriate bearing stratum is not readily accessible [in the instance of a 3 in (7.5 cm) minipile, this depth must be no more that about 18 ft (5.4 m)]; and (c) structural loads are high (over about 60 kN or 6.5 tons).

2. In soft clay or loose granular soils, no contribution to load design is assigned to shaft friction. In some cases, the shaft friction is assigned a negative number, i.e., soil consolidation or surcharge load.



(c)

FIGURE 7B.4.9. (c) The obvious and serious rake in yet another instance of failed steel minipiles. This scene shows two minipiles exposed by a bockhoe.

3. In silty or sandy soils, the load-bearing capacity of the pile can diminish with time because of movement of water between soil particles (collapsing soil).
4. Raked piles are more susceptible to failure because of lateral stress (i.e., heave in clay soils) than those that are truly vertical. A rake angle of 10° is most common but must not exceed 15°. Refer also to Section 7B.8.5.
5. Minipiles (high length/diameter ratio) often fail more in strength as a column as opposed to failure in soil support.
6. Straightness, true vertical alignment, and concentric loading are useful or necessary to prevent buckling of the pile.
7. Pile lengths should be limited to no more than 75 times their diameter. For a 3 in (7.5 cm) diameter pile, the maximum effective length should be 18.75 ft (5.6 m).

FIGURE 7B.4.10. This photograph illustrates one of the problems that can occur as a result of using the perimeter beam as the resistance to drive a steel minipile. Not only have the bolts sheared (losing all support for the structure), but the foundation beam has also been seriously sheared.

8. A single, central rebar is useless for resisting lateral forces (bending). It does, however, provide additional resistance to tensile forces.
9. Casing provides only minimal resistance to bending of minipiles. Caged rebar is the best solution, but the pile or pier diameter must be such as to permit proper placement of concrete.
10. Prior to underpinning, all voids beneath the floor slab must be grouted (mudjacked). Note: When underpinning is performed in conjunction with raising the foundation, the author prefers that the mudjacking be performed subsequent to raising.
11. Minipile spacing is normally limited to about 5 ft (1.5 m) maximum.
12. Piles larger than approximately 6 in (15 cm) are normally augered.
13. A safety factor between 2½ and 3 is suggested.

The foregoing tends, to some extent, to repeat certain of the facts presented in prior paragraphs. However, the 13 issues cited are quotes from the referenced publication. Each item is quite essential to minipile performance and therefore viewed as quite important to the use of these methods for underpinning residential foundations. Quite obviously, the steel minipile was not designed or intended for use in expansive soils.

7B.4.4.1.3 Costs. Aside from the other problems inherent to minipiles, is their inordinate cost—often 1.8 to 4 times the price for a conventional 12″ diameter (30 mm) drilled concrete pier—must be considered. Perhaps the *only* saving grace lies with the fact that exterior (perimeter) piles can be installed more quickly and with somewhat less damage to the landscaping. However, bear in mind that the cost differential between a single piling and concrete pier is sufficient to purchase a pick-up load of landscape plants. Is the idea of shortening the time for repairs by a day or so worth the sacrifice? Another *seeming* advantage might be the fact that some of the minipile contractors

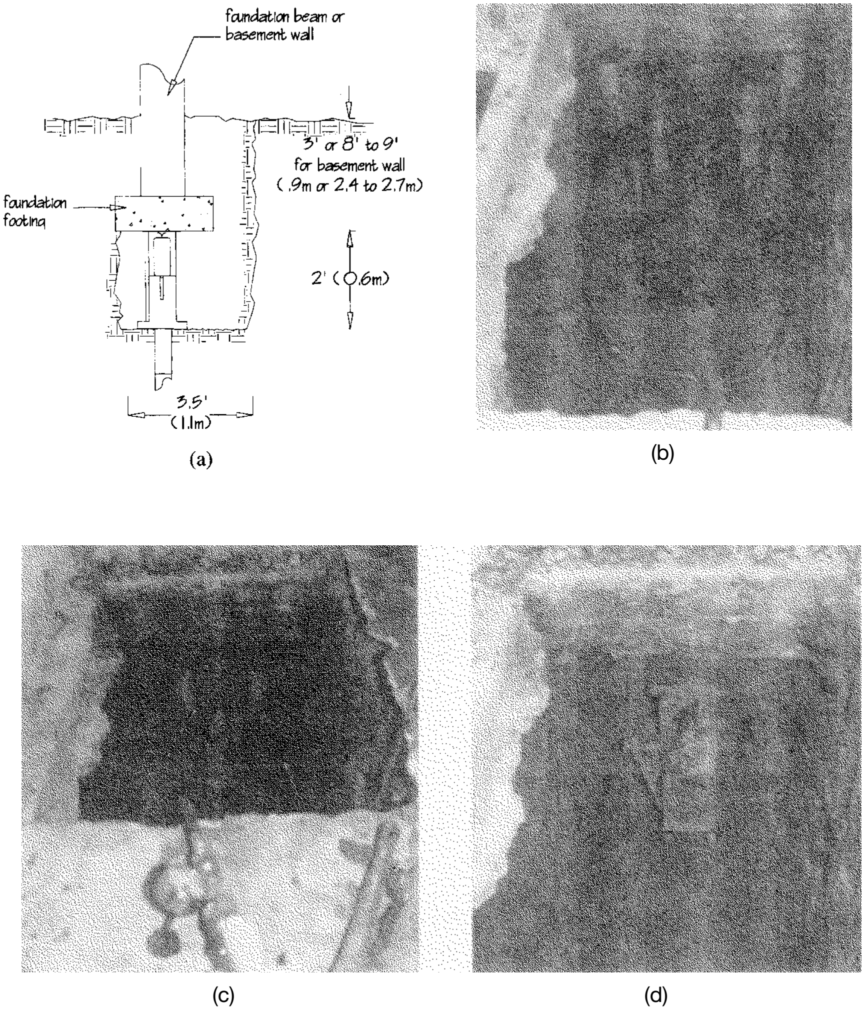


FIGURE 7B.4.11. Concentric pile technique. (a) Drawing of equipment in place; (b) photo of equipment in place; (c) photo of beam being raised; (d) beam and pier locked in place.

offer lifetime warranties. Anytime you are offered one of these, solicit legal advice. You may find words but questionable protection (see Section7C).

The cost for steel driven pilings varies significantly. This variation is brought about by: 1) differences in technique, 2) franchise or royalty fees, 3) differences in excavation costs (deeper perimeter beams or footings require increased excavation, which in some cases might extend to depths at which shoring is required), 4) geographic locations, 5) local codes, and 6) closer spacing (nominally the smaller diameter underpins are located on 3–6 ft (0.9 to 1.8 m) centers. The costs range from \$450 to \$1500 (the higher cost was supplied by *PBF Magazine*, May 15, 1997 in the article "How Deep Will They Go").

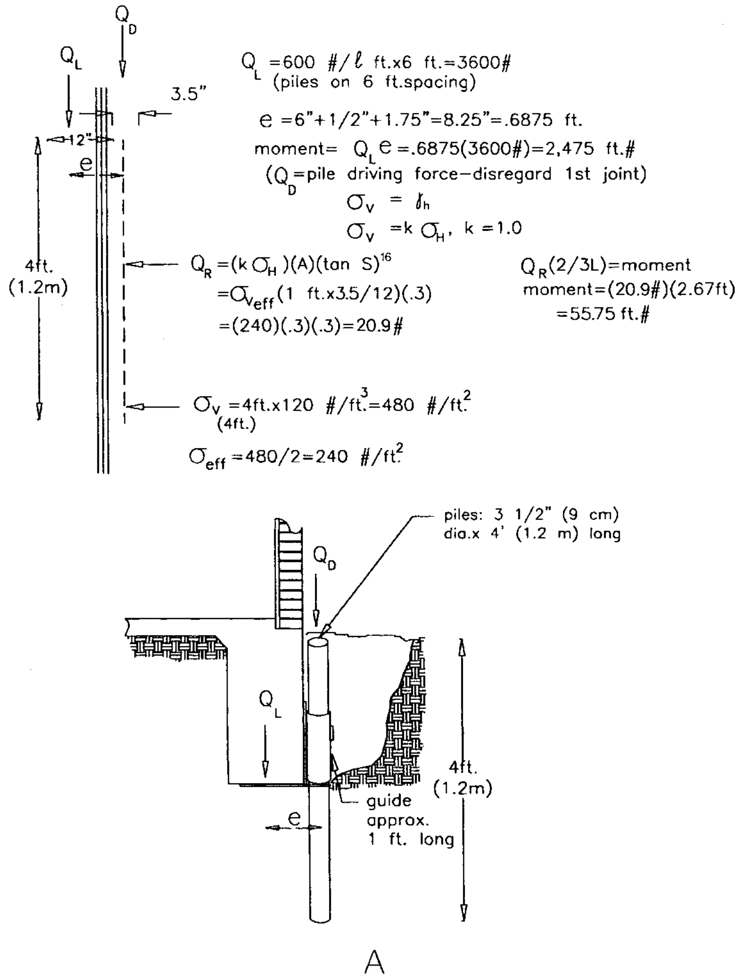
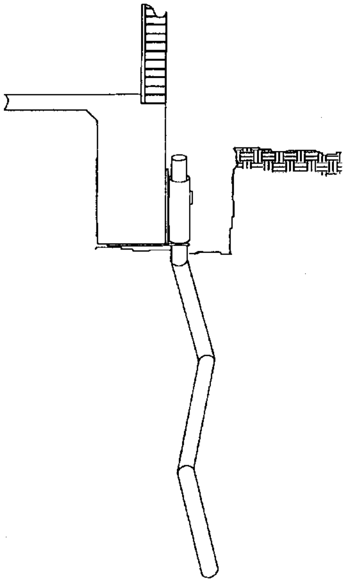


FIGURE 7B.4.12. Eccentrically driven piles.

7B.4.4.2 Ultraslim Concrete Piers

Whether hydraulically driven or poured in place, the ultraslim concrete piers are also not generally reliable as underpinning options.[16,17] This is particularly true when they are used in areas with expansive soil.

7B.4.4.2.1 Hydraulically Driven Concrete Cylinders. Hydraulically driven concrete cylinders, nominally 12″ (30 cm) in length and 4″ to 6″ in diameter (10 to 15 cm) (whether strung on a central cable or merely stacked), are prone to failure in lateral stress. If it becomes a concern, they also have virtually no resistance to tensile stress, particularly in those instances where the cable is not stressed. Figure 7B.4.13 is an artist's rendering of this pile system. When the method was first introduced, the procedure was to drive the concrete test cylinders into the ground beneath the perimeter beam. The cylinders were stacked on top of each other and the driven length of cylinders was finally referred to as a "pier."



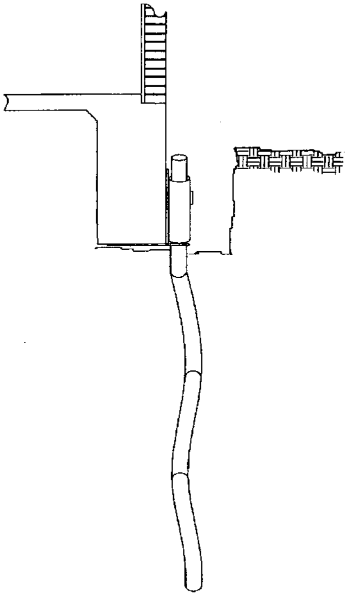


FIGURE 7B.4.12. (*continued*). Eccentrically driven concrete piles.

This requires a real stretch of the imagination. Next, seemingly to overcome the criticism of lack of alignment and total lack of tensile or lateral resistance to stress, the cylinders were strung on a single ⅜″ (0.09cm) posttension cable. This is somewhat comparable to digging a 6 in (15 cm) diameter fence posthole to some depth, inserting a single #3 (⅜″ or 0.9 cm) rebar, pouring concrete and representing the result as a "structural pier." The difference being due to the singular fact that the cable provides more resistance to shear than the rebar. A 270k ⅜″ diameter cable provides shear resistance of about 22,950 lb, provided it is tensioned. A #3 rebar (40,000 psi) provides a resistance of only 4400 lb. Could anyone be convinced that this would work? Let's bring back the Montana

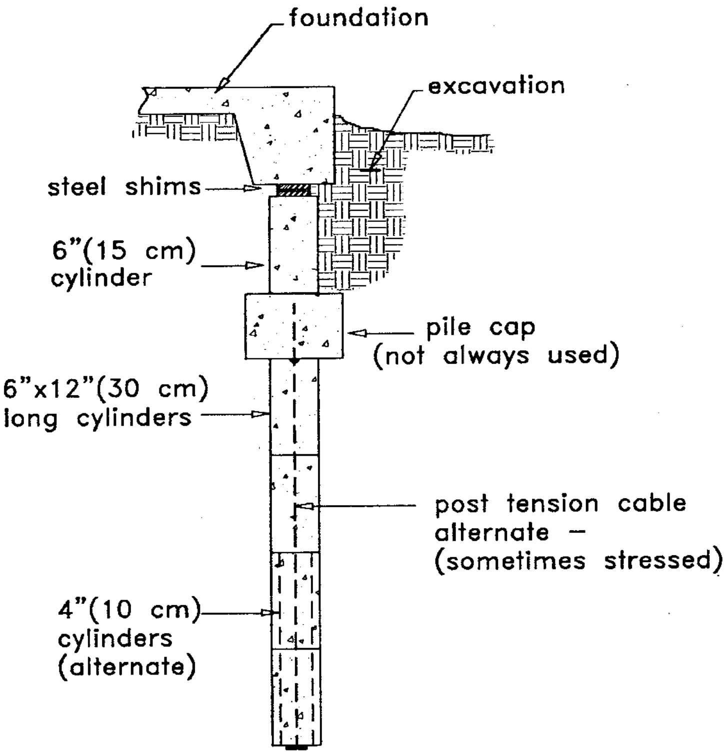


FIGURE 7B.4.13. Hydraulically driven concrete pilings.

oceanfront property. In fact, this method appears to have little market value under any circumstances.

The cost to the consumer for these underpins is reportedly in the range of $225.00 to $350.00 each. However, due to the closer spacing, the effective "job" cost will increase proportionately.

7B.4.4.2.2 Ultraslim Piers (Poured Concrete). Usually 8 in (10 to 20 cm) in diameter and nominally 5 to 6 ft (1.5 to 1.8 m) on centers, the drilled ultraslim piers suffer problems due to stress failures (both tensile and lateral). Adequate steel reinforcing can provide some desired resistance to these stresses. However, the small diameter makes the proper placement of both concrete [1½ in (3.75 cm) minus aggregate] and steel most difficult.[16,17] Figure 7B.4.14 illustrates the problem. With the steel in place, the clearance is only 1.875 in (4.76 cm) (this can vary some with the manner in which the steel is caged.) A safe passage for 1½″ (3.75 cm) rock requires a minimum clearance of 4.5 in (11 cm). [Proper clearance for concrete placement is generally assumed to be three times the diameter of the largest size aggregate. In the case of concrete with 1½″ minus aggregate, the desired clearance would be 3 × 1.5″ or 4.5″ (11 cm).] Concrete technology also suggests a 3 in (7.6 cm) coverage of concrete between steel and steel or steel and form. The three (#3s) practically preclude the placement of regular hard rock concrete. Concrete strength must be sacrificed by either opting for a pea gravel concrete or decentralized steel. Neither option is acceptable. [Conventional piers often use as many four #3s (0.95 cm), #4s (1.27 cm), or #5s (1.6 cm). With either, the steel can be tied in a 6″ (15 cm) cage, which allows ample clearance for concrete.]

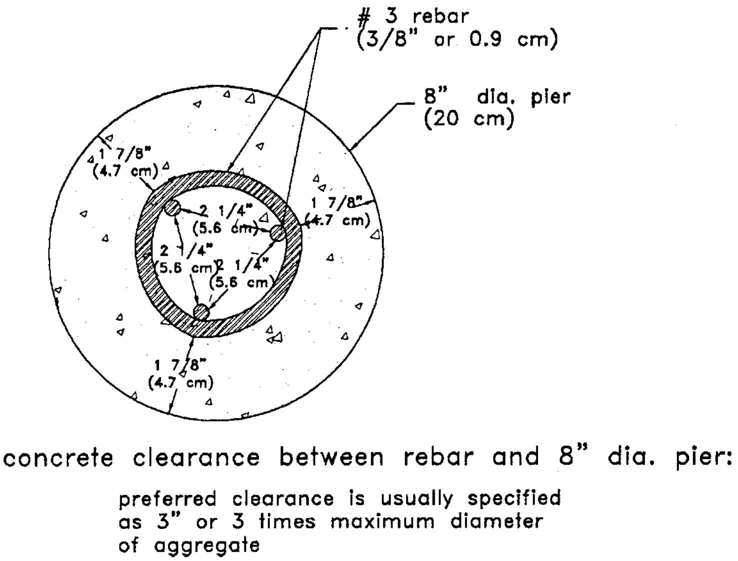


FIGURE 7B.4.14. Poured concrete slim pier.

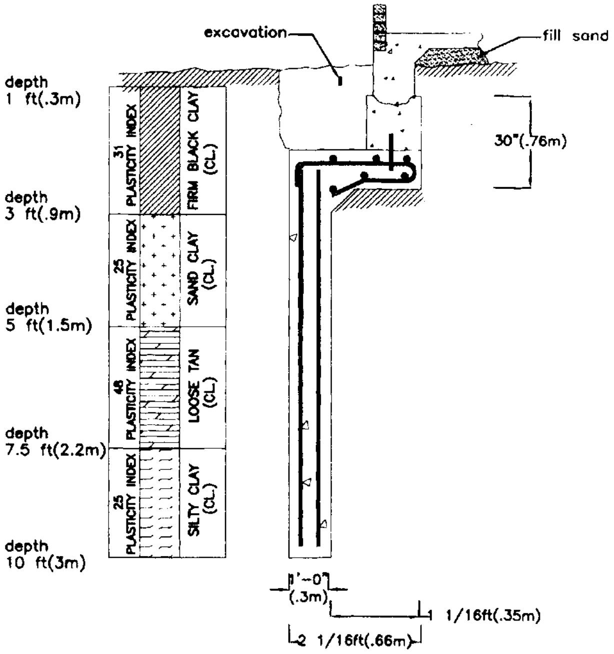
The 8 in (20 cm) diameter piers are often used in tandem to presumably offer bearing support comparable with conventional 12 in (30 cm) diameter concrete piers. In linear strength calculations, a single 12″ (30 cm) diameter concrete pier (without steel reinforcement) is equivalent to 2.25 8 in (20 cm) diameter piers. The big difference between the two is noted when: a) the piers are subjected to eccentric loading or lateral stress or b) steel rebar interferes with concrete placement.

Dual ultrapiers have been suggested. The geometrics of the tandem installation causes the individual piers to be raked, often well in excess of 20°. The recommended rake for any load-bearing pier is 10 to 15°.[16,17,91] Also, the dual piers do not overcome the obstacles cited for the single ultraslim. Refer to Section 7B.4.4.3 and Figures 7B.4.14 and 7B.4.15. In the case of slab foundations, proper mudjacking (following the underpinning) might somewhat overcome the inadequacies of the ultramini piles.

7B.4.4.2.3 Cost. As opposed to the steel minipiles, either of the ultraslim concrete piers are less costly than conventional concrete piers or spread footings, often in the range of $200.00 to 275.00 each. A contractor who charges $200 each for the ultraslim drilled pier should enjoy much higher profit than a contractor who, under similar conditions, installs a 12 in (0.3 m) pier for $300.00 to $350.00. The cost to put in an 8 in (20 cm) pier is less than half the cost necessary for the larger pier.

7B.4.4.3 Combination Spreadfootings and Drilled Pier

On occasion, piers or pilings are used in conjunction with the spreadfooting (haunch). Here the theory is to utilize the best support features of each design in the hope of achieving a synergistic effect. (Also, as a practical matter, a haunch is always necessary to provide a base from which to raise the beam.) In practice, the goal is not always attained. When soil conditions dictate the spreadfooting, the pier provides little, if any, added benefit. The integration of the deep pier as part of the spreadfooting, generally, has no deleterious features, provided: 1) the cross-sectional area of the footing is not diminished, 2) the pier does not penetrate a highly expansive substratum that has access to water, 3) the diameter of the pier is at least 10 in (25 cm), and 4) the pier is not



A

FIGURE 7B.4.15. (A) Conventional pier.

raked to an excessive degree. Consider Figure 4B.5.15. Water in contact with the CH clay at a 5 to 7½ ft (1.5 to 2.3 m) depth could cause the piers to heave, whereas the spreadfooting would be stable. Refer to "The Effects of Soil Moisture on the Behavior of Residential Foundations in Active Soils", R. W. Brown and C. H. Smith, *Texas Contractor*, May, 1980. Certain steps are available to avoid or minimize friction (upheaval) in the design of shallow piers. These efforts include: 1) using the "needle" or "slim" pier or piling (reduced surface area), 2) belling the pier bottoms (usually effective with conventional diameter shafts), 3) placing a friction-reducing membrane between the pier and the sidewall of the hole.

When dual piers are used, several alterations are required. First, the pier diameters are generally limited to about 8″ (20 cm). Second, the dual piers must be raked (deviated from vertical). This rake should never exceed 12°, but in practice often exceeds 20°.[37] The small diameter pier introduces all the drawbacks inherent to that described in Section 7B.4.4.2.2. The cost for these piers is equivalent to, or exceeds, that for the conventional 12″ (0.3 m) concrete piers, but obviously is less effective.

7B.4.4.4 Hydro Piers

The so-called hydro pier is too weak in both theory and practice to merit discussion. Maintaining a constant level of soil moisture is certainly a beneficial control for expansive soil movement. However, water injection alone is not likely to provide any degree of beneficial leveling and can cause serious damage.[3] This method does not really produce a "pier," hence the label is misleading. Figure 7B.4.16 is a drawing intended to show the placement and development of a typical hydro pier. The principal claims seem to be:

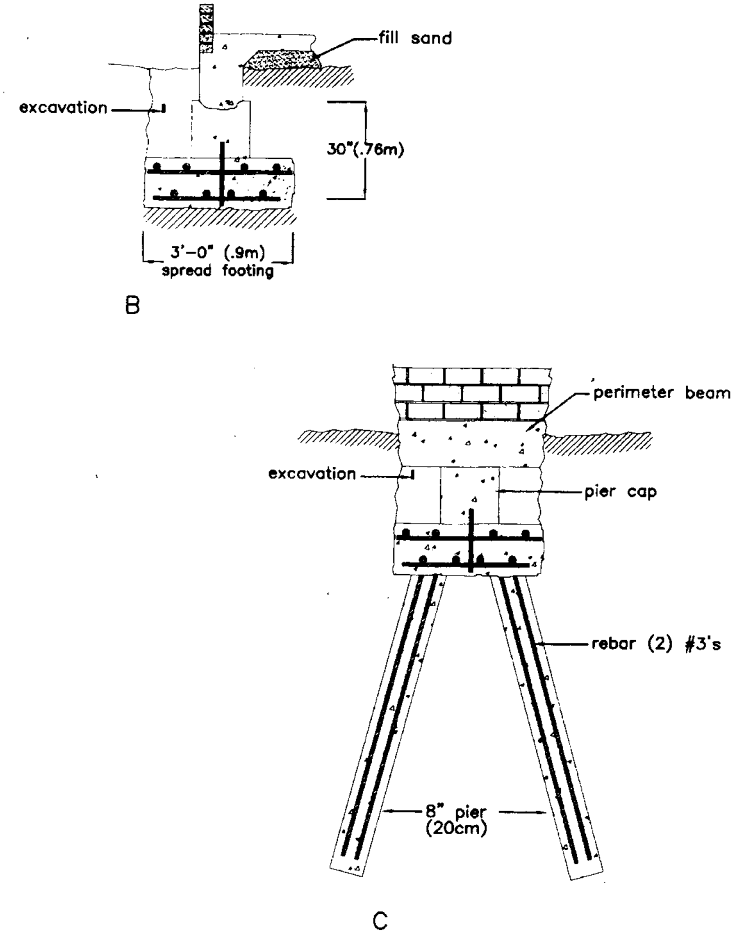


FIGURE 7B.4.15. (*continued*). (B) Spreadfooting; (C) dual minipier.

1. The system continuously supplies water to the soil, thus preventing settlement.
2. Another company using an identical process advertises "uniform foundation raising" of up to 3″ (7.5 cm).
3. Some users claim that the vertical weep hose delivers water that expands the clay adjacent to the hose, thus creating a "hydro pier."
4. Others claim that the expansion of the clay adjacent to the hose constricts the hose to the extent that at some point water flow is shut off.

No one can take this seriously.

7B.4.4.5 Underpinning an Interior Slab versus Mudjacking

Reference to either the BRAB book[36] or the PTI manual,[90] both of which deal with the design of residential slab on grade foundations, will not reveal a single design dependent upon piers or any other

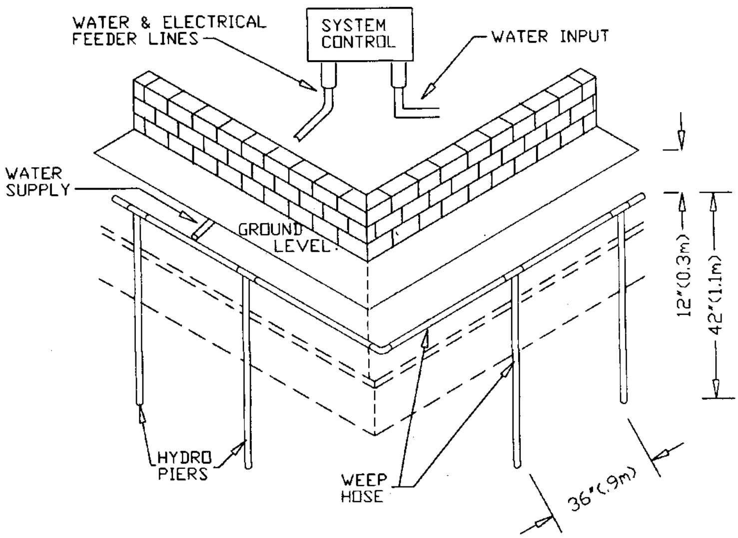


FIGURE 7B.4.16. Typical installation of hydropiers.

form of underpinning. It is true that both manuals make reference to special conditions wherein piers/piles might be incorporated into the foundation design. This, however, represents a "special" exception. In both manuals it is clear that the basic foundation design depends upon essentially 100% on support of bearing soil. Again, as an exception, the design can be "beefed" up to accommodate conditions requiring minor cantilever for bridging. In fact, the BRAB manual also provides for a *structural* slab that is designed for self-support independent of the shallow surface soils.

Perimeter Underpinning. Consistent with the basic slab design, piers are not normally considered as appropriate supports for the perimeter of the basic slab foundation. However in remedial operations piers are sometimes *required* to augment the leveling requirements. In so doing, care must be taken: 1) not to create a situation that causes the beam to bridge a distance greater than its design will safely accommodate, and 2) to restore the required "soil support" as quickly as feasibile. The latter is normally accomplished by mudjacking. Experience over 40 years has established that a safe spacing for the underpins is about 8 ft. The piers merely augment the raising operation. The mudjacking is called upon to support the foundation as it had been intended.

Case Law can be found that seemingly supports this position. In O'Donnell vs Bullivant, 933 sw. 2d 754 w, Texas Appellant Court FWT, defendant underpinned a slab foundation and the court found "In effect the pilings altered the foundation of the O'Donnell residence so that it's weight rested on much deeper soil than was originally designed. In other words the design, construction and installation of 32RB pilings altered the type foundation for the O'Donnell residence from a surface soil supported reinforced concrete slab to a slab supported on deep perimeter piles." (Mudjacking after installation of pilings would have negated this concern.)

Interior Underpinning. Although normal residential slabs are *seldom* designed to accommodate perimeter piers, they are virtually *never* designed for interior underpinning. The interior support beams are less substantial than the perimeter beam hence less adaptive to bridging support. The interior slab is designed for virtually zero bridging. Very few practicing engineers or competent foundation repair contractors will recommend the installation of interior slab underpins. This is for the obvious reasons mentioned in preceding statements. Plus, if the underpins are to be installed from within the residence, entry must be provided by breaking out sections of the existing slab floor. As a rule each area broken out will approach or exceed 5 to 6 ft^2. Multiply this by the number of interior underpins that are to be installed and the total area of removed floor is quickly realized. The slab excavation is accompanied by excessive intrusion through the slab foundation and followed up by floor patches that are never equivalent in strength to the undamaged floor. At the conclusion of this underpinning the entire slab foundation must be mudjacked. (Tunnelling is an alternative to breaking through the floor slab. This procedure is even more expensive, more dangerous to personell, and represents even a greater structural threat than does the break-out approach. Refer to the discussion or perimeter underpins.)

The sensible solution for raising the interior slab is simply mudjacking. This procedure has been successfully used for well over 40 years and has involved hundreds of thousands of foundations.

In rare instances, even the most conscientious contractor or engineer might suggest interior piers. This could occur in instances where, for example, interior fireplaces have settled to the extent that cosmetic options are not viable. The heavy weight represented by the fireplace, surrounded by a weak 4″ slab, is not conducive to mudjacking. Rather than lift the heavy fireplace, the grout is likely to raise (hump) the surrounding, weak floor slab. In this case, underpinning is justified.

Figure 5.17 depicts an engineer's rendering for restoring an interior slab floor to (or near) original grade. The 28 "dots" represent pier locations to be installed by breaking out sections of the slab floors. (It is the author's opinion that a slab foundation is to be broken only in extremely rare occasions, i.e., for raising an interior fireplace.) The installation of the interior piers: 1) threatens the structural integrity of the slab, 2) creates a horrendous mess, and 3) is inordinately expensive. On this particular job, the owner alluded to a bid of "well over $12,000.00" to install the piers/pilings and fill voids. A bid for competent mudjacking was less than $4,000.00. The job was mudjacked to full satisfaction of owner with little inconvenience to the tenants and no significant damage to the floor slab.

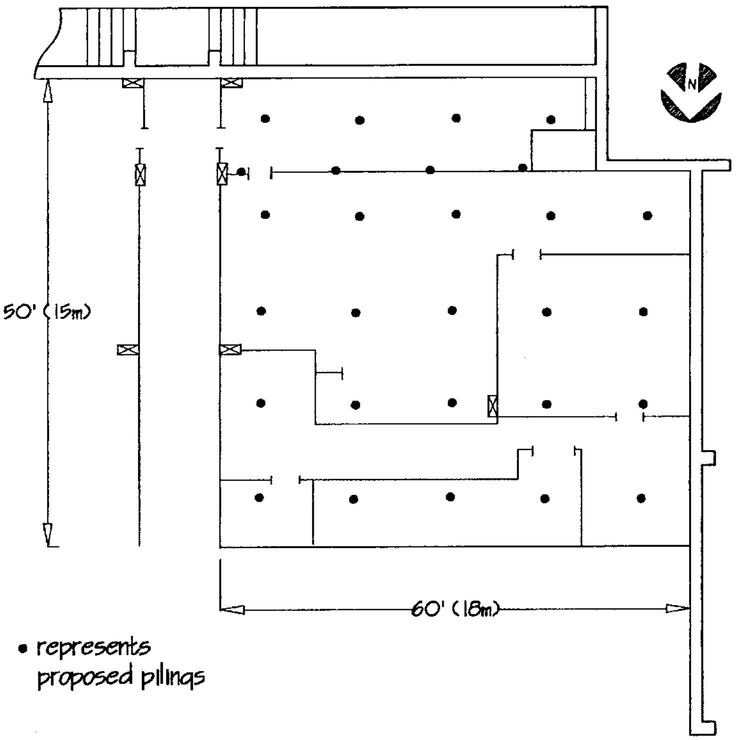


FIGURE 7B.4.17. Underpinning a slab foundation versus mudjacking.

7B.4.5 Summary

Regardless of the type of underpinning support, two precautions must be exercised. First, concrete should be poured into the pier shafts or pads as quickly after excavation as possible. Ideally, concrete would be poured the same day as excavation. Second, no steel or wood should be exposed below grade either as shim or pier materials. Exposure to water and soil will corrode the steel or rot the wood within an unusually short period of time. Third, the pier cap should be poured in place to ensure intimate contact between the pier and the irregular bottom of the perimeter beam. Shims or flat surfaces in contact with the irregular surface of the beam will result in either immediate damage to the concrete beam or subsequent resettlement due to the same effect over a period of time, both due to crushing of concrete protrusions. Fourth, concrete placed into shafts deeper than about 15 ft (4.5 m) should be tremmied.

Also, bear in mind that none of the underpinning techniques attempt to "fix" the foundation to prevent upward movement. This is by design, to avoid subsequent, uncontrolled damage to the foundation and emphasizes the fact that if shallow, expansive soils are subjected to sufficient water, the soil expansion will raise the foundation off the supports.

7B.4.6 Design Tables for Concrete

Tables 7B.4.1 through 7B.4.4 should provide the reader a ready correlation for slump, concrete composition, and relative strengths, as well as weight and size of rebar. These tables should be self explanatory. Table 7B.4.5 gives the amount of concrete required to pour at various diameters. Table 7B.4.6 offers the effective prestress provided by different sized posttension cables.

The following example calculates the amount of concrete that should be ordered to pour 15 piers, 12 in (0.3 m) in diameter, to a depth of 20 ft (6 m). From Table 7B.4.5:

$$15 \times 0.029 \text{ yd}^3/\text{ft} \times 20 \text{ ft} = 8.7 \text{ yd}^3 \ (6.6 \text{ m}^3)$$

The following calculates the amount of concrete that should be ordered to pour 15 piers, 18 in (0.45 m) in diameter, to a total depth of 18 ft (5.4 m), bell pier shafts to 36″ diameter (0.9 m).

$$15 \times 0.065 \text{ yd}^3/\text{ft} \times 18 \text{ ft} = 17.55 \text{ yd}^3 \ (13.3 \text{ m}^3)$$

$$\text{Extra for bell} = 15 \times 0.255 = 3.85 \text{ yd}^3 \ (2.9 \text{ m}^3)$$

$$\text{Concrete required} = 17.55 + 3.85 = 21.4 \text{ yd}^3 \ (16.26 \text{ m}^3)$$

Table 7B.4.7 provides a table to assist with the cursory selection of safe bearing areas required to support various loads on soils of a given unconfined compressive strength. Consider the following example. Given a soil with an unconfined compressive strength of 1000 lb/ft^2 (157 kN/m^2) and an

TABLE 7B.4.1 Recommended Slumps for Concrete

	Slump, in (cm)	
Types of structure	Minimum	Maximum
Massive sections, pavements, and floor laid on ground	1 (2.54)	4 (10.16)
Heavy slabs, beams, or walls; tank walls; posts	3 (7.62)	6 (15.24)
Thin walls and columns; ordinary slabs or beams; vases and garden furniture	4 (10.16)	8 (20.32)

TABLE 7B.4.2 Mixture for 1 yd^3 (0.76 m^3) of 3000 lb/in^2 (21 MPa) Concrete

Material	Amount/sack of cement	Total amount/yd
Cement (5 sacks)	94 lb (42.5 kg)	470 lb (212 kg)
Sand	314 lb (142.5 kg)	1570 lb (712 kg)
Coarse aggregate	345 lb (157 kg)	1725 lb (784 kg)
Water	7 gal (max) (26.5 L)	35 gal (132.5 L)

TABLE 7B.4.3 Reinforcement Grades and Strength

	Minimum yield strength, f_y, lb/in^2 (MPa)	Ultimate strength, f_u, lb/in^2 (MPa)
Billet steel		
Grade 40	40,000 (276)	70,000 (483)
60	60,000 (414)	90,000 (620)
75	75,000 (517)	100,000 (690)
Rail Steel		
Grade 50	50,000 (345)	80,000 (552)
60	60,000 (414)	90,000 (620)
Deformed wire		
Reinforced	75,000 (517)	85,000 (586)
Fabric	70,000 (483)	80,000 (552)
Cold-drawn wire		
Reinforced	70,000 (483)	80,000 (552)
Fabric	65,000 (448)	75,000 (517)

TABLE 7B.4.4 Weight, Area, and Perimeter of Individual Bars*

Bar dimension #	Unit weight		Diameter, d		Cross section area (CSA)		Perimeter	
	lb/ft	kg/m	in	cm	in^2	cm^2	in	cm
2	0.167	0.249	0.250	0.635	0.05	0.32	0.786	2.0
3	0.376	0.560	0.375	0.95	0.11	0.71	1.178	2.99
4	0.668	0.995	0.500	1.27	0.20	1.29	1.571	3.99
5	1.043	1.560	0.625	1.59	0.31	2.0	1.963	4.99
6	1.502	2.24	0.750	1.9	0.44	2.84	2.356	5.98
7	2.044	3.05	0.875	2.22	0.60	3.87	2.749	6.98
8	2.670	3.99	1.00	2.54	0.79	5.1	3.142	7.98
9	3.400	5.07	1.125	2.86	1.00	6.45	3.544	9.0
10	4.303	6.41	1.250	3.175	1.27	8.19	3.990	10.13
11	5.313	7.92	1.375	3.49	1.56	10.06	4.430	11.25
14	7.65	11.40	1.750	4.45	2.25	14.52	5.32	13.51
18	13.60	20.26	2.250	5.715	4.00	25.8	7.09	18.0

*The ultimate yield for a #3 bar (grade 40) would be 0.11 in^2 × 70,000 psi or 7700 lb_f (7.7 kips).

TABLE 7B.4.5 Concrete Piers*

	CSA		Volume per linear foot			Volume 2× bell		
	ft^2	m^2	ft^3	yd^3	m^3	ft^3	yd^3	m^3
10″ dia.	0.54	0.05	0.54	0.02	0.015			
12″ dia.	0.785	0.073	0.785	0.029	0.022	1.18	0.044	0.033
14″ dia.	1.07	0.099	1.07	0.0396	0.03			
16″ dia.	1.4	0.13	1.4	0.052	0.04			
18″ dia.	1.77	0.164	1.77	0.065	0.05	6.9	0.255	0.194
24″ dia.	3.14	0.29	3.14	0.116	0.088	18.84	0.698	0.53

*Pier diameters smaller than 10 in (25 cm) should not be considered for underpinning foundations.

TABLE 7B.4.6 Posttension Cables, Effective Prestress, kips

Strand diameter, f_{pu}(in ksi)	Strand area (in^2)	(0.7)(f_{pu})(Aps)* (kips)	Average prestress loss (kips)†	Effective prestress (kips)
3/8-270K	0.085	16.10	1.3	14.8
7/16-270K	0.115	21.70	1.7	20.0
1/2-270K	0.153	28.90	2.3	26.6

Assumed prestressed losses of 15 ksi, actual losses should be calculated as per Section 7B.5.6 in reference 90. Note that the effective strand CSA is less than that for a comparable deformed bar. For example, the CSA of a #3 rebar is 0.11 in, whereas that for the ⅜ cable is 0.085. The effective area is the actual combined cross section areas of the individual strands making up the cable. The term f_{pu} is equivalent to f_u in Table 7B.4.3. In practice, the process of tensioning the cables is associated with several losses in tensile strength. More on this can be found in the PTI manual.[90].

*The factor 0.7 compensates for stress losses in the cable immediately after anchoring.

†(15 ksi)(CSA, in^2).

TABLE 7B.4.7 Maximum Load-Bearing Capacity (Q) lb, (N)*

		Unconfined Compression Strength of Soil (q_u), lb/ft^2 (N/m^2)					
Support	Bearing area $ft^2/(m^2)$	1000 (4448)	2000 (8869)	3k† (13,344)	4k (17,792)	6k (26,688)	8k (35,584)
3″ dia.	0.05 (.0045)	50 (20)	100 (40)	150 (60)	200 (180)	300 (120)	400 (160)
4″ dia.	0.09 (.008)	90 (35.6)	180 (71)	270 (107)	360 (142)	540 (214)	720 (284)
6″ dia.	0.20 (.018)	200 (80)	400 (160)	600 (240)	800 (320)	1200 (480)	1600 (640)
8″ dia.	0.35 (.032)	350 (142)	700 (284)	1050 (426)	1400 (568)	2100 (852)	2800 (1136)
10″ dia.	0.55 (.05)	550 (222)	1100 (444)	1650 (666)	2200 (888)	3300 (1332)	4400 (1776)
12″ dia.	0.785 (.070)	785 (315)	1570 (630)	2355 (945)	3140 (1260)	4710 (1890)	6280 (2520)
1 ft^2	1.0 (.090)	1000 (414)	2000 (826)	3000 (1242)	4000 (1626)	6000 (2484)	8000 (3312)
16″ dia	1.4 (.13)	1400 (578)	2800 (1156)	4200 (1734)	5600 (2312)	8400 (3468)	11,200 (4624)
18″ dia	1.76 (.16)	1760 (711)	3520 (1423)	5280 (2133)	7040 (2847)	10,560 (4266)	14,080 (5693)
2 ft^2	4 (.36)	4000 (1601)	8000 (3200)	12k (4800)	16k (6400)	24k (9600)	32,000
2.5 ft^2	6.25 (.56)	6250 (2500)	12.5k (5000)	18.75 (7500)	25k (10,000)	37.5k (15,000)	
3 ft^2	9.0 (.81)	9000 (3603)	18k (7266)	27k 10,809			

*The values given in the table do not include a margin of safety. The indicated total safe load would be divided by the appropriate safety factor to give the safe design load. For Example: A soil with a q_u of 2000 lb/ft^2 would require a minimum bearing area (CSA) of 1 ft^2 to handle a weight load of 2000 lb, again without regard to any safety factor.

†k = 1000 lb.

intended load of 4000 lb (1818 kg). None of the conventional piers would safely accommodate this load in end bearing alone. However, a 12″ (.3 m) pier with a 2 ft × 2 ft (0.372 m^2) haunch would carry the load. Again, this approach is intended for screening purposes only. Any concerns should be resolved by a complete mathematical analysis based on the specific and complete project specifications. Suggestions provided by the Table 7B.4.7 are generally quite conservative.

At the end of the day, the structural load must be accommodated by the soil. The foundation as well as any underpinning merely represent devices used to distribute structural loads to perfect a soil advantage. A simple review of the weight-bearing capacity of a typical soil is represented in the following analysis.

Figure 7B.4.18 depicts a typical poured concrete pier beneath the perimeter beam of a slab foundtion. Refer also to Figure 7B.4.1. Assume a uniform load on the perimeter beam equivalent to 600 lb per linear foot. The piers are to be located on 8 ft centers. The structural weight distributed to each pier location would then be Q_w = 600 plf × 8 ft = 4800 lb. The piers are assumed to be vertical.

The resistance to this load provided by the soil can be summarized as follows:

Pier end-bearing only. Assume q_u =3 tsf, π_4 = 0.785, and A = 0.785D^2

$$Q_{EB} = 0.785 \text{ ft}^2 \times 3 \text{ tsf}$$
$$= 2.36 \text{ tons} = 4720 \text{ lb}$$

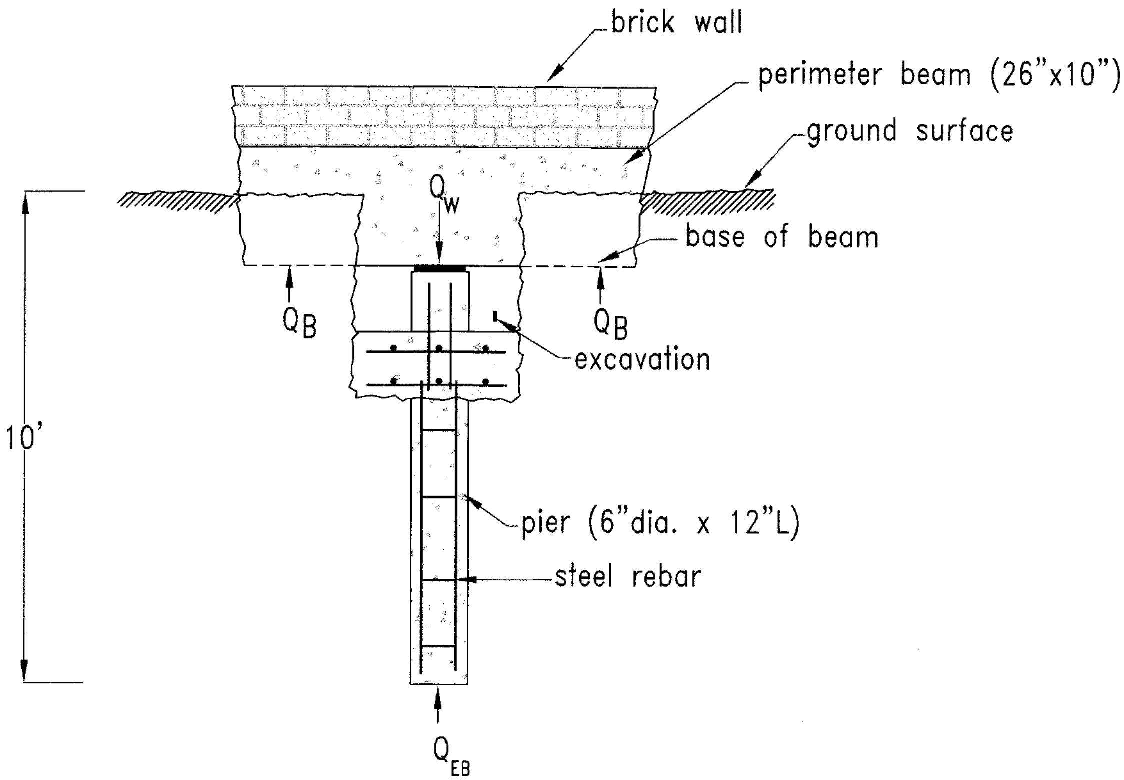


FIGURE 7B.4.18. Soil load capacity.

Resistance provided by haunch:

$$
\begin{aligned}
Q_H &= 2.5 \text{ tsf} \times (5.0 - 0.985) \text{ ft}^2 \\
&= 2.5 \text{ tsf} \times 4.215 \text{ ft}^2 = 10.54 \text{ tons} \\
&= 21080 \text{ lb}
\end{aligned}
$$

Weight capacity of perimeter beam in full contact with soil or subsequent to mudjacking:

$$
\begin{aligned}
Q_s &= (0.833 \text{ ft} \times 1 \text{ ft})(2 \text{ tsf}) \\
&= 1.666 \text{ tons per linear foot of beam}
\end{aligned}
$$

Assuming an 8 ft increment

$$
\begin{aligned}
Q_x &= 3332 \text{ lb/in} \times (8 - 1) \text{ ft} \\
&= 3332\ (7) = 23{,}324 \text{ lb}
\end{aligned}
$$

At this point the maximum load support for the soil in question is:

$$
\begin{aligned}
Q_{\max} &= Q_{EB} + Q_H + Q_S \\
&= 4720 + 21080 + 23324 \\
&= 49{,}124 \text{ lb}
\end{aligned}
$$

The factor of safety under the prevailing conditions would be:

$$SF = 49124/4800 = 10.23$$

If the pier skin friction were to be considered, the soil bearing capacity would increase by:

$$Q_{\text{friction}} = (K\overline{\sigma}_v \tan S)\, A_{\text{surface}} \qquad \overline{\sigma}_v = \gamma_{\text{soil}}\, \text{D}$$

Refer to references 12, 15–17. Assume $\gamma = 125$ lb/ft^3, tan $S = 0.45$[12,15–17]

$$
\begin{aligned}
Q_{\text{friction}} &= (1)(7)(125) + (12)(125)/2(.045)(5 \text{ ft})(\pi)(1 \text{ ft}) \\
&= (875 + 1500/2, \text{ lb ft}^2)(0.45)\ (15.75 \text{ ft}^2) \\
&= (1187.5 \text{ lb})(0.45)\ (15.75) \\
&= 8416 \text{ lb}
\end{aligned}
$$

NOTE: Neglect top 7 ft of pier depth for friction calculations.[12,15–17] The total combined soil bearing now becomes 49,124 + 8416 or 57,540 lb. The new safety factor becomes 54570/4800 = 12. As an aside, consider a 6″ diameter pressed pile. Refer to Figure 7B.4.13. There is no haunch to consider.

Pier end bearing only: $q_u = 3$ tsf

$$
\begin{aligned}
Q_{EB} &= (0.785)(6''/12'')^2 \times 3 \text{ tsf} \\
&= (0.196 \text{ ft}^2)\ (3 \text{ tsf}) = 0.589 \text{ tons} \\
&= 1177.5 \text{ lb}
\end{aligned}
$$

This pier (soil, actually) would not carry the structural load of 4800 lb or, for that matter, 3000 lb if the pier spacing were reduced to 5 ft. The nature of the pressed pile is such that no design benefit can be attributed to skin friction. Probably, the pier could function if enhanced by the load capacity provided by the perimeter beam. Proper mudjacking to insure intimate soil–beam contact would be a must. (Q_s is also referred to as Q_b). Therefore:

$$
\begin{aligned}
Q_S + Q_{EB} &= 23{,}324 + 1{,}177.5 \\
&= 24{,}501
\end{aligned}
$$

Under these conditions, the factor of safety could become:

$$SF = 24501/4800 = 5.1$$

The end bearing capacity of an 8″ diameter pier under the same conditions would be:

$$
\begin{aligned}
Q_{EB} &= 0.785\ (8/12)^2 \text{ ft}^2 \times 3 \text{ tsf} \\
&= 0.35 \times 3 = 1.05 \text{ tons} \\
&= 2093.3 \text{ lb}
\end{aligned}
$$

This pier would not carry the structural load without help. The usual 4 ft^2 haunch would provide some help. For example:

$$
\begin{aligned}
Q_H &= 2.5 \text{ tsf} \times (4 - 0.35) \text{ ft}^2 \\
&= 2.5 \text{ tsf } (3.65) \text{ ft}^2 = 0.125 \text{ tons} \\
&= 18{,}250 \text{ lb}
\end{aligned}
$$

Added to the end bearing the total unit bearing, capacity becomes:

$$
\begin{aligned}
Q_{EB} + Q_H &= 18{,}250 + 2093 \\
&= 20{,}343 \text{ lb} \\
SF &= 20343/4800 = 4.2
\end{aligned}
$$

The skin friction would also provide additional load capacity.

The same or a similar analysis can be used to screen potential designs for underpins. Sometimes, simple variations in design features can be incorporated to enable the desired factor of safety.

7B.4.6.1 Spacing Underpins for Raising Perimeter Beams

Table 7B.4.8 presents the design factors for perimeter beams of the designated size. The following example problems present the design checks for a nonreinforced concrete beam 10″ × 16″ spanning distances of 8, 10, and 12 ft. The assumed live load was 500 plf. This particular design was selected because it is the about the lowest size recommended for use in areas with expansive soils. Note that this beam passes all checks for the spans studied. This suggests that in underpinning, the piers/piles *could* be spaced up to 12 ft apart. In the real world this is not likely to happen because the problems associated with actual foundation leveling are not generally symmetrical. Note also Section 9A for field-selected spacings. In practice, the repair contractor would be responsible for selecting the spacing best fitting his technique and economy, so long as the spacing did not exceed the safe limit of the existing beam.

7B.4.6.1.1 Example Calculations. The following examples relate to a 10″ × 16″ unreinforced concrete beam supported on 8 ft (2.4 in), 10 ft (3 m), and 12 ft (3.6 m) spans. The stress areas checked are bending moment (M), workable stress (f_B), deflection (Δ) and shear (V). Stability of the beam (D/b) is also shown. This beam is theoretically capable of a 12 ft span under the assumed parameters.

Bending Moment/Workable Stress (f_B) [$W_s = w = 1.4\ (1.66.5) + 850 = 1083$ plf, continuous beam with three or more continuous support points. Refer also to Section 7B1.8]

10″ × 16″ beam (0.833 ft × 1.33 ft × 150 lb/ft^3 = 166.5 plf)

8ft span
$$
\begin{aligned}
M &= wl^2/\ 120 \text{ in-lb} = 1083(96)^2/120 = 83{,}174 \text{ in-lb} \\
f_B &= M/S_X = 83{,}174/506.2 = 163 \text{ psi} \\
F_B &= 1350 \text{ psi}
\end{aligned}
$$

TABLE 7B.4.8 Concrete Design Factors*

Dimension	CSA, in^2 (ft^2)	W_D, plf	I_X, in^4	S_X, in^2	F_B, psi	F_V, psi	E, psi
10″ × 16″	160 (1.11)	166.8	3,413	506.2	1350	220	3×10^6
10″ × 20″	200 (1.39)	208.5	6,664	666.7			
10″ × 24″	240 (1.67)	250.5	11,515	960			
10″ × 30″	300 (2.08)	312.9	22,491	1500			

$W_s = 1.4 \times W_s + 1.7 \times W_L = 1.4 \times W_D + 1.7(500) = 1.4 \times W_D + 850.$
$f_c = 3{,}000$ psi, $F_V = 4\sqrt{f_c}$, $F_B = 0.45\, f_c$.
*Structural, LL = 500 plf.

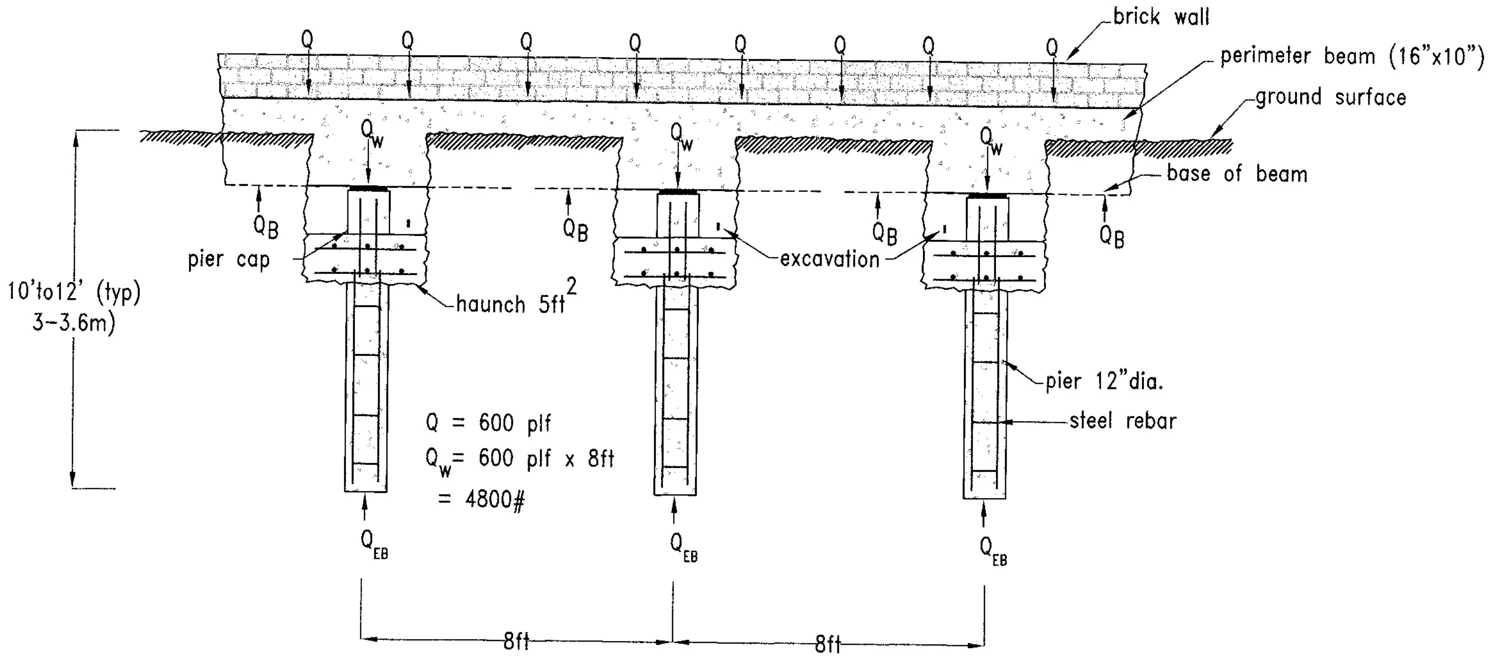


FIGURE 7B.4.19. Load evaluation of concrete beam during underpinning. Note: During the raise, jacks are set on the haunch and used to lift the beam. At, or near, the conclusion of the raise, the concrete beam is not likely to have contact with the supportive soil ($Q_B = 0$). (This contact is ultimately restored by mudjacking in the event of slab foundations.) The pier caps are removed and the excavation back filled to complete the underpinning operation. [Steel pipes (1¼ in I.D.) are frequently used to temporarily support the beam to allow for both the removal of the jacks and the proper cure of the concrete pier caps.]

10 ft span $M = 1083(120)^2/120 = 129{,}960$ in-lb
$f_B = 129{,}960/506.2 = 256.7$ psi

12 ft span $M = 1083(144)^2/120 = 187{,}142$
(144′) $f_B = 187{,}142/506.2 = 369$ psi
$F_B = 1350$ psi

Deflection Check (Δ)
10 ft span $\Delta = wl^4/1743\ EI_x$ (E and I_x from Table 7B.4.5)
$(120)^4 = 207.36 \times 10^6$
$= 1083(120)^4/1743(3 \times 10^6)(3413) =$
$\Delta = (1083)(207.36 \times 10^6)/(1743)(3 \times 10^6)3413 = 0.01'' < 0.267''$
$\therefore$ OK in deflection

Stability Check:

D/b = 16/10 = 1.6 < 2 $\therefore$ OK

12 ft span $(144)^4 = 429.98 \times 10^6$

(144″)
Deflection (Δ) continued

$\Delta = wl/1743(E)(I_x)$ (E and I_x from Table 7B.4.8)
$= (1083)(144)^4/(1743)(3 \times 10^6)3413$
$\Delta = 0.028$ in $< 0.267''$
$\therefore$ OK in deflection

Shear Check

8 ft span Shear $V = 5wl/96$, $f_V = 3/2$ (V/CSA)

(96″) $V = (5)(1083)(96)/96 = 5415$ lb
$f_V = 1.5$ (5415 lb)/160 in^2 = 50 psi (from Table 7B.6.4.8 $F_V = 220$)
50.8 < 220 $\therefore$ shear OK

10 ft span $V = (5)(1083)(120)/96 = 6768.8$ lb

(120″) $f_V = (1.5)(6{,}768.8)$lb/160in^2 = 63.45 psi
63.45 psi < 220 (F_V) $\therefore$ shear OK

12 ft span $V = 5wl/96 = 5(1083)144/96$
$= 8122.5$ lb
$f_V = 1.5(8122.5)/160$ in^2 = 76.15 psi
76.15 < 220 $\therefore$ shear OK

7B4.7 Unconfined Compressive Strength of Soils

Foregoing sections have discussed the effects of distribution of various load factors to bearing soil. This section will relate those loads to the unconfined compressive strength of various soils.

7B4.7.1 Slab on Grade Foundations

Tthe bearing capacity of most undisturbed cohesive soils is sufficient to carry the working loads for normal residential or light construction. For example, a 600 plf load carried by a beam 12″ wide requires an allowable soil bearing capacity (q_a) of only 600 psf (Table 7B.4.9A. The laboratory-measured unconfined compressive strength (q_u) with a safety factor of three would also be about 600 psf. Refer to the Terzaghi equation (in Section 7B4.7.3). It follows that if a slab foundation is properly mudjacked concurrent with underpinning, the bearing capacity of the underpin is not a material concern. Other issues, such as underpin heave, needs appropriate evaluation. When dealing with disturbed native soil, or other specific nonconforming conditions, the use of piers (or other special support features) are generally incorporated into the original slab foundation design. Otherwise, the design of the slab requires basically 100% support by the bearing soil (exclusive of underpins), and piers are not incorporated into the design.

7B4.7.2 Pier-and-Beam Foundations

Especially when void boxes are used, the foundation support is relegated entirely to the piers. In this instance, the design of any repair underpins must consider the specific foundation and soil conditions. Design features for repair underpinning should address such factors as:

1) Structural load
2) Spacing with underpins
3) Load capacity of the underpins being considered. (Such factors as load-bearing capacity, shear resistance, resistance to lateral shear, potential heave, and longevity are principal aspects to be considered.)
4) Costs
5) Geotechnical data (seldom available)

Table 7B4.9, for piers, gives the structural load (less SF) for various assumed load values. For example, a load of 500 plf (Q) on a 10″ wide perimeter beam requires an allowable bearing capacity (q_a) of 520 psf. (In table 7B4.9.B, extrapolate between the 400 plf and 600 plf values.) For piers (or other supports) on 8 ft centers, the structural load is 8 × 500 or 4000 lb. A 12″ diameter pier with 4000 lb load requires a soil with allowable unconfirmed compressive strength (q_a) of 5100 psf (from Table 7B4.9.A). Assuming a SF of 3, the lab determined bearing capacity (q_u) to provide this allowed capacity would become 5100/1.235 psf or 4130 psf. (This is calculated using the Terzaghi and Peck equation, which simplifies to $q_a = 3.7\ q_u/\text{SF}$ or $q_a = 1.235\ q_u$ for round or square footings, when incorporating a SF of 3.0.) The analysis thus far discounts any benefit for side wall friction. This will be discussed in the following paragraph. For a safety factor 2.0, q_u becomes 4130 × ⅔ or 2750 psf. If the soil at the desired depth cannot accommodate the required load, the pier can be belled. With a 24″ bell, the required bearing (q_a) is reduced to only 5100/4 or 1575 psf. (The bell also reduces or eliminates the problem of pier heave.) However, when not required for structural needs, the bell adds $50.00 to $150.00 per pier. This might increase the per pier cost by something like 15% to 45%. An engineer specifying the bell should consider Texas Engineering Board Rule 131.151(a), which states in part: ". . . is the notion than an engineer is to provide an optimized, cost effective design."

Most pier designs (for remedial, purposes) incorporate the use of a haunch. This pad can be counted on to increase the bearing capacity of the pier shaft. This feature will be discussed further in the following paragraph. Also refer to Sections 7B4.2.2 and 7B4.3.

7B4.7.3 Steel-Reinforced Piers

There is little doubt that the 12″ steel-reinforced pier is recognized as the optimum underpin. Refer to the references 17 and 44. The point of concern focuses on the specific design of the pier. Design features will be addressed as follows:

1. The appropriate depth depends upon the specific soil characterists. However, for the Dallas Metroplex, a study of several hundred geotechnical reports suggests that the optimum depth is in the range of 10 ft. In the real world, neither repair contractors nor forensic engineers normally have access to foundation plans, much less geotechnical data. Costs make it impractical to require this information as a basis for providing a routine repair proposal or forensic report.
2. Pier shear suggests that the minimum reinforcing is two #3's. The local preference among contractors is 4 #3's or #4's. The consensus of engineers in the Metroplex is 4 #4's. Reinforcement beyond this is overkill for normal construction conditions. An occasional exception might occur wherein exceptional strength might be required to control tensile or lateral stresses.

TABLE 7B4.9 Allowable Soil Bearing Capacity Required q_a, psf *

A						
Beam width	Applied structural load, Q, plf					
in (ft)	400	600	800	1000	1500	2000
24″ (2 ft)	200	300	400	500	750	1000
18″ (1.5 ft)	300	400	600	750	1125	1500
12″ (1.0 ft)	400	600	800	1000	1500	2000
10″ (0.83 ft)	480	720	960	1200	1800	2400
8″ (0.66 ft)	600	900	1200	1500	2250	3000
6″ (0.5 ft)	800	1200	1600	2000	3000	4000

B						
Pier	Applied structural load (Q), lb/pier					
Dia.; ft CSA	2,000	3,000	4,000	5,000	6,000	10,000
2 ft (3.1 ft^2)	650	975	1,300	1,625	1,950	3,250
1.5 ft (2.25 ft^2)	1,175	1,760	2,350	2,940	3,525	5,875
1 ft (0.785 ft^2)	2,550	3,825	5,100	6,380	7,650	12,750
0.83 ft (0.54 ft^2)	3,700	5,550	7,400	9,250	11,100	18,500
0.66 ft (0.34 ft^2)	5,880	8,820	11,760	14,700	17,640	29,400
0.5 ft (0.2 ft^2)	10,000	15,000	20,000	25,000	30,000	50,000
0.3 ft (0.07 ft^{2}2)	28,600	42,900	57,200	71,500	85,800	142,000

*For strip beams, the table values for q_a are approximately equivalent to the desired unconfined compression strength q_u of the soil, with a built-in safety factor of three. Refer to note 2 of the Terzaghi equation (Section 7B.4.3). Assuming a 600 plf load on a 10″ wide beam, the required allowable bearing capacity of the soil would be 720 psf. This would require an unconfined compressive strength q_u of 720 psf with a safety factor of 3.0. For a safety factor of 2.0, the required q_u would be 720 × ⅔ or 480 psf.

For square or round piers, q_u can be determined by dividing the q_a value by 1.235. The resulting q_u has a built-in factor of three.[99] Assume a structural load of 500 plf supported by 10″ diameter piers on 8 ft centers. The piers load (Q) is 8 × 500 or 4000 lb. These conditions would require an allowable soil bearing capacity q_a of 7400 psf. This would convert to a measured unconfined strength q_a of 7400/1.235 or 6000 psf with a safety factor of 3.0. For a safety factor of 2.0, the q_u becomes 6000 × ⅔ or 4000 psf. If the pier is designed with a 5 ft^2 haunch, the effective load becomes 4000 lb/5 ft^2 or 800 psf. The q_u required would again be 650 psf with a safety factor of 3.0 or 435 psf with a safety factor of 2.0. The 10″ diameter pier requires a q_u of 1.45 times greater than for a 12″ pier with the same Q. This by no means suggests a pier larger than 12″ diameter.[44]

3. Pier spacing for the 12″ diameter pier is generally a nominal 8 ft.
4. The potential advantage for belling the pier shafts were touched on in prior discussions.
5. The application of side wall friction as a design issue is a bit uncertain. Generally, the depth of the SAZ is excluded from side wall friction calculation. In the Metroplex, this depth is perhaps 7 ft; however, 86% of natural soil moisture variations occur at a depth of about 3 ft.[102,103] As a compromise, the top 5 ft might be discounted. Other factors such as heat condition of the pier itself might influence greater depths. For these reason, the influence of skin friction is often arbitrarily discounted for the top 5 to 7 ft of the pier depth.[91] Often, the actual pier depth starts some 3 ft or so below the ground surface; refer to Figure 7B4.1. [The effective depth of the soil active zone (SAZ) is also measured from the surface.] Skin friction (the beneficial kind) can be readily estimated and added to the end-bearing capacity of the pier or pile. For normal residential repair, it is probably acceptable to discount skin friction as a viable design factor unless geotechnical data is available that dictates otherwise. Refer to Section 9A for more discussion on underpinning. For repair purposes the haunch provides more additional bearing capacity than would skin

friction; refer to Figure 7B4.1. (Generally, the base of the haunch is below the depth of *principal* soil activity; in the example it is approximately 40″ or 3.3 ft.) Typically, the unconfined compressive strength (q_u) for shallow soils is less than that for soils at greater depths. However, the given haunch distributes the assumed structural load (4000 lb) over 5 ft^2, resulting in a load of 800 psf. The load on a 12″ pier would then be 800 × 0.785 ft^2 or a mere 630 lb, requiring a soil with an allowed compressive strength of about 800 psf. (The value determined by extrapolation using Table 7B4.9B and the value for 2000 lb or 2550 psf.) With a safety factor of 3, the unconfined soil-bearing strength required would be 800/1.235 or 650 psf. For a safety factor 2, q_u becomes 650 × ⅔ or 435 psf.

Again, these calculations are simplified but should prove to be adequate for repair estimations and/or evaluations. For a more professional approach, refer to references 16, 17, 77, 91, 99, 100, and 105.

The Terzaghi–Peck equation[99] is:

$$q_a = (0.95)q_u(1 + 0.3\ \mathrm{B/L})$$

Where:

q_a = allowable bearing capacity
$q_a = 3.7\ q_u/\mathrm{SF} = 1.235\ q_u$, with a safety factor of 3.0 (3.7/3 = 1.235)
or $q_u = q_a/1.235$
q_u = average unconfined compressive strength
B = width of footing (a 1 on 1 ratio is used for round or square footings)
L = length of footing

1. $q_a = (0.95)\ (1.3)\ q_u = 1.235\ q_u$, with a safety factor of 3.0
2. $q_a = (0.95)\ (1.3\ B/L)\ q_u$; when L is very much larger than B, the product of (0.95) (1 + 3 B/L) becomes approximately 1.0. Then $q_a = q_u$, with a SF of 3 "built-in."

7B4.7.4 Conclusions

The allowable bearing capacity (q_a) of the soil for required various structural loads were used to determine workable values for unconfined compressive strength (q_u). This is necessary to provide the engineer with a design factor that can be established from laboratory tests. Following paragraphs will further relate the "theoretical" (q_u) to the real (q_u) values measured for soils within the Dallas–Fort Worth Metroplex.

7B4.7.5 Slab Foundations

In slab foundations, loads are transferred by the perimeter beam directly to the soil. The results of 62 geotechnical reports involving soils within the DFW *prior to construction* and concerning the depth 1–2 ft indicate:

1. The average (q_u) for soils at a depth of 1–2 ft was 5515 psf.
2. Not a single test produced q_u results less than 1000 psf.
3. Only 3.0 % (2) of the tests gave q_u results less than 1500 psf.
4. Only 8.0% (5) reported q_u values between 1500 and 2000 psf.

As stated earlier, the soil in the Metroplex is capable of safely supporting the loads imposed by normal residential construction. In fact, from Table 7B4.9.A, a beam 12″ wide can accommodate a load of 1000 plf with a safety factor of 3. Generally, slab foundations are designed without piers, except for fire places or unusual site conditions. Piers (or any other soil of the underpin) would not be a structural asset to leveling a normal slab foundation.

However, in the real world, piers are often advised to handle remedial problems related to upheaval—which, by the way, accounts for well over 70% of all repairs to slab foundations in the

DFW Metroplex. In fact, the addition of piers to an existing, normal slab, breaches the basic design. As a rule, slab foundations are neither designed nor intended to be supported intermittently by piers but by soil over effectively 100% of its area. If follows then, that where a slab must be underpinned during the repair phase, this practice must be followed by proper mudjacking.

7B4.7.4.2 Underpinning the Beam

In underpinning the beam, loads are essentially transferred from the perimeter beam to piers or other underpins. The q_u value for soils at the 3–4 ft depth taken from geotechnical reports, again prior to construction, covering 52 different Mapsco areas and 95 individual tests indicate that:

1. an average (q_u) of 6060 psf.
2. zero values less than 1000 psf.
3. 4% (4) less than 1500 psf
4. 4% (4) between 1500 and 2000 psf.

The q_u value for soils in the areas at depths of 9–11 ft involving 135 different tests show:

1. an average q_u of 30,540 psf
2. zero tests for q_u less than 1000
3. 7% (11) q_u values less than 2500 psf
4. 18% (24) q_u values between 2500 and 4000 psf
5. 10% (14) q_u values between 4000 and 5000 psf
6. 10% (13) q_u values between 5000 and 6000 psf

A straight shaft 12″ in diameter with an applied structural load of 4000, 5000, and 6000 lb requires a q_u of 5100/1.235 (4100 psf), 6380/1.235 (5170 psf), and 7650/1.235 (6200 psf), respectively. The computed values for q_u contain a “built-in” safety factor of 3.0. Refer to Table 7B4.9.A. For soil tested with q_u values less than that required to safely accommodate the intended load, some adjustment is required. This could involve merely decreasing the span, belling the shaft, considering the bearing capacity provided by a haunch, or reducing the safety factor. Belling the shaft is an option that should be considered only on an “as necessary” basis, due to costs. Other than the cost issue, belling the pier shaft carries little or no other baggage. Reducing the safety factor to 2.0 would reduce the corresponding q_u values to 4100 × ⅔ (2730 psf), 5170 × ⅔ (3445 psf), and 6200 × ⅔ (4130 psf). A 5 ft^2 haunch will reduce the loads (q_a) to 4000/5 (800 psf), 5000/5 (1000 psf), and 6000/5 (1200 psf), respectively. Again, divide the reduced loads by 1.235 to determine the q_u value for the corresponding q_a. The resulting values will again have a “built-in” safety factor of 3.0. In the past, the possibility of pad (or “mushroom”) heave provided some concern; however, current thoughts more or less discount that possibility.[26,27] This is due, in part, at least, to the facts that the pads are heavily loaded (as compared to the perimeter beam), the pads are located below the *principal* soil active zone, and, in the case of the haunch, the integral pier shaft serves as an anchor. It is far more likely that the perimeter beam raises off the pier cap. Refer to Figures 7B4.7 and 7B4.6. The concern over pad settlement is also not realistic. The spreadfooting (9 ft^2) distributes the load over such an area that, for all practical purposes, virtually any cohesive soil can safely carry the load. The haunch (5 ft^2) affords similar advantages, plus the pier shaft serves as additional support.

7B.5 BASEMENT OR FOUNDATION WALL REPAIR

7B5.1 Introduction

The remedial approaches to basements do not fit in with “normal” foundation repair procedures. This introduces a “special case” scenario, which is of concern only in certain locales. Basement construction is desirable in instances where:

1. The source for heat needs to be lower in elevation than the living area. Heat rises.
2. Land costs are high, making it less expensive to excavate than to spread out.
3. A frost line, or permafrost, must be considered.

In the modern-day South, few basements are built or have been built during the last 60 years. Due in large part to the this, the author's experience with basement repair is limited. The following examples were taken from this limited exposure plus repair procedures designed by structural engineers (PEs) over the country.

7B.5.2 Typical Approaches to Basement Repairs

Our first repair example involves the construction of a new wall inside an original wall. This is sometimes referred to as a "sister" wall.[13] The structural load is ultimately transferred to this addition. No effort is made to plumb or reinforce the existing basement foundation wall. Refer to Figure 7B.5.1a. The sequence of construction is essentially as follows:

1. Shore existing floor joists to lessen or remove the structural load from the defective wall.
2. Sand-blast or scarify surface of existing wall to remove latence (loose concrete material) and provide an improved bonding surface.
3. Break out floor slab (and protruding strip footing where applicable) to prepare for installation of supporting concrete piers and beam (where applicable). (Typically, for a wall height less than about 10 ft (3 m) with constructions loads less than about 1500 lb per linear foot (2268 kg/m), the piers could be 10 to 12 in diameter (25–30 cm), spaced on approximately 8 ft (2.4 m) centers.
4. Place steel lintels across keyways to hold fascia brick. An alternative would be to remove brick.
5. Break out concrete or remove sufficient concrete block to create keyways of approximately 1 × 2 ft (0.3 × 0.6 m), 6 ft (1.8 m) on centers (OC).
6. Drill dowel holes into existing foundation/basement wall. Often the rebars would be no. 7's (⅞ in or 2.2 cm), spaced to create a pattern on the order of 12 to 18 ft^2 (1.0 to 1.5 m^2) in area.
7. Place steel and pour piers and beam.
8. Place steel and pour sister wall. Often the concrete is placed through the keyways by concrete pumps.

Note: This and following examples are meant to be general. Specific loads and job conditions will dictate the size of reinforcement and spacing of the support members. Also, virtually every installation will benefit from some type of waterproofing.

Figure 7B.5.1b presents yet another problem. This approach permits some plumbing of the defective wall, and is generally more effective with masonry wall construction than with concrete. The wall materials should suffer little or no deterioration. The general procedure for this installation is:

1. Excavate fill adjacent to wall exterior.
2. Drill holes through basement/foundation wall to facilitate take-up bolts. Generally, the bolts would be 1 to 1½ in (2.54 to 3.8 cm) in diameter, located at least three to a tier and 3 to 6 ft (0.9 to 1.8) OC.
3. Place channel iron on each side of wall. Install bolts and tighten.
4. Waterproof wall as necessary and back fill with a gravel hydrostatic drain system.
5. As with virtually all attempts to plumb a failed basement/foundation wall, it is often desirable to supplement the system with additional force to push the wall into plumb. Refer to Figure 7B.5.1c.

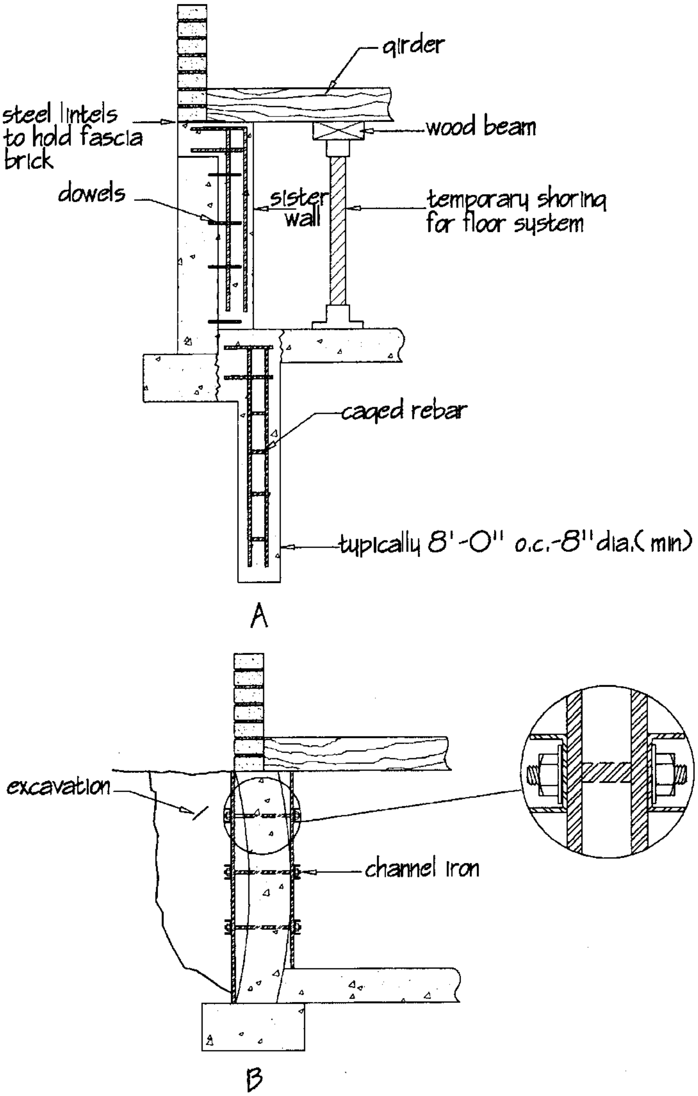
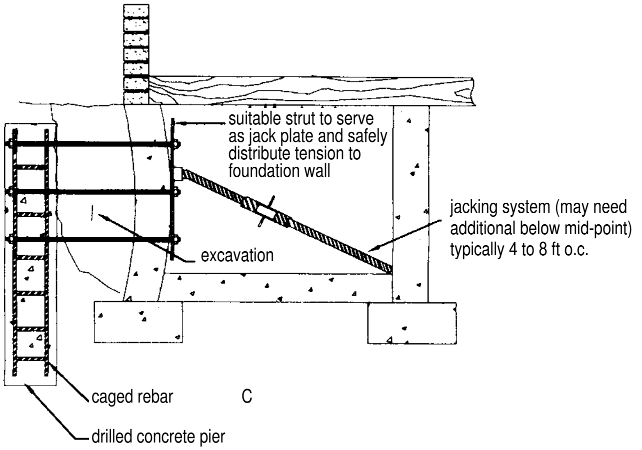


FIGURE 7B.5.1. Basement/foundation wall repair. (a) Failure in foundation (basement) wall, transfer or load; (b) failure in foundation wall, plumbing desired.

Another technique, more suited to actually plumb a concrete basement/foundation wall, is depicted in Figure 7B5.1c. This technique uses an opposing wall to secure the jacking system, which is utilized to plumb or align the basement/foundation wall. Depending on such factors as existing wall design, load conditions, and degree of rotation, a second battery of jacks may be required. Typically, each battery of jacks would be placed 4 to 8 ft (1.2 to 2.4 m) apart. A typical sequence for the installation of this system would be



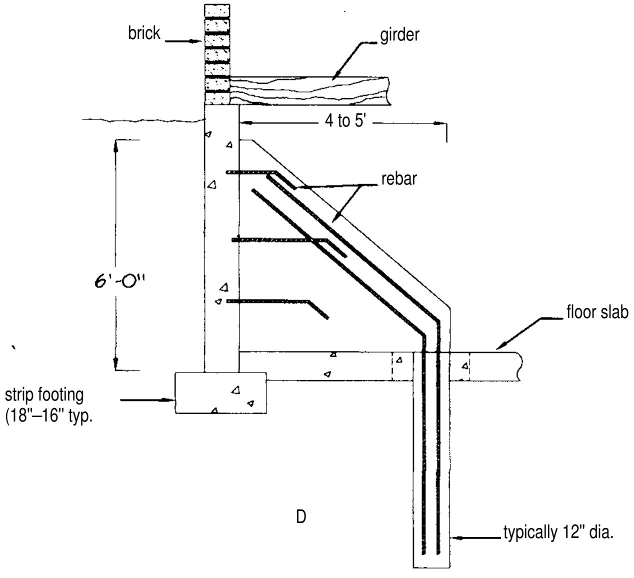


FIGURE 7B.5.1. (*continued*). Basement/foundation wall repair. (c) Failure of basement wall, plumbing desired; (d) knee brace to control rotation.

1. Drill holes for dywidag (or similar) bars, 1½ to 2 in (3.8 to 5.0 cm) in diameter, through basement/foundation wall. Locate bar in external drilled pier shaft. Holes are typically three to each pier, 4 to 8 ft (1.2 to 2.4 m) OC.
2. Drill and pour the external concrete piers.
3. Excavate behind the existing wall. (Alternatively, the back fill could be excavated prior to placement of the pier. This would necessitate forming the piers but would allow the stripfooting to be

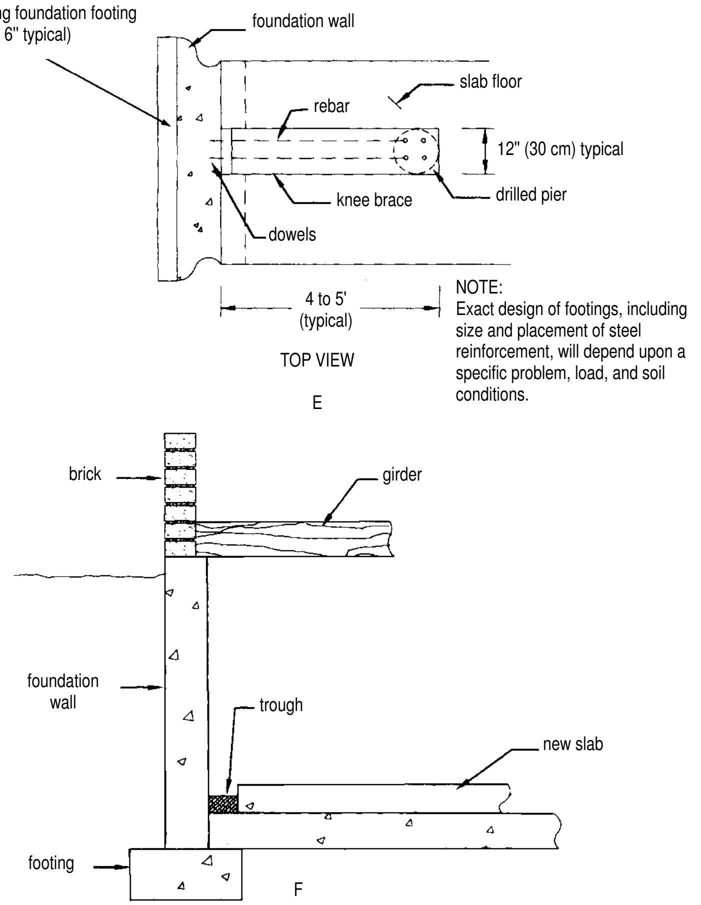


FIGURE 7B.5.1. (*continued*). Basement/foundation wall repair. (e, f) Water control.

broken out at pier locations. Removal of the protruding stripfooting would allow contact of the pier to the existing wall with vertical alignment.)

4. Place channel iron and commence jacking operation.
5. Waterproof wall as required and back fill with suitable gravel hydrostatic drain.

The knee brace is yet another approach to retrofit basement/foundation walls. This method is fairly simple and adequate to sustain wall rotation. This method is not intended to accomplish any degree of vertical alignment of the *existing wall*. Figure 7B.5.1d depicts the design of a typical knee brace. Depending on specific conditions, a second pier may be required immediately adjacent to the existing wall. However, for lightly loaded conditions, a schedule of dowels will prevent any slip between the wall and brace. Considering normal basement heights [less than 10 ft (3 m)] and lightly loaded conditions, the placement of the braces might be 6 to 10 ft (1.8 to 3 m) on centers.

NOTE: The knee brace is also frequently used to control outward rotation of foundation walls in deck-high construction. The primary limitation would be instances where the defective wall is situated on a "zero lot line." When the foundation walls are 5 ft (1.5) high or less and the wall structure permits, the placement of the supports might be as far apart as 10 to 20 ft (3 to 6 m). For certain minor problems, particularly those associated with lightly loaded conditions, an adequate restoration approach could be to utilize the jacking system illustrated in Figure 7B.5.1a to raise and level the floor joist system. Permanent supports would consist of supplemental beams (wood as a rule) supported atop lally columns. Generally, the columns [often 4 to 5 in (10 to 15 cm) steel pipe] would be fitted with steel plates top and bottom. Frame basement walls are particularly suitable for option. Job conditions control the number, design, and placement of these supports.

7B.5.3 Hadite Block Walls

Other relatively minor problems sometimes involve hollow concrete block (Hadite) walls. If the intent is to strengthen the wall or shut off minor water seepage, the problem might be addressed by filling the blocks with a concrete mix. Normally, at least two rows of injection holes are drilled through the inside of concrete block wall. Typically, the lowest row would be about 4 ft (1.2 m) off the floor and a second row near the top of the wall. Adjacent (lateral) holes are used to ensure complete penetration of all voids. Initially, the holes might be approximately 4 ft (1.2 cm) OC. If any question arises concerning filling all voids, intermediate holes can be drilled. The lowermost row of holes is normally injected first and may or may not be allowed to attain initial set before the top row is pumped. The stage pumping reduces the hydrostatic load on the lower courses of block.

7B.5.4 Basement Water Infiltration

With persistent water infiltration, the solution shown in Figure 7B.5.1e might be acceptable. The water that penetrates the wall is collected in the trough at the perimeter of the floor slab. Water is then transported to an adequate sump or drain. A slight variation with this is to: 1) construct a sump in the floor slab and 2) build a screed floor over the concrete slab. The sump accumulates the water and pumps it to a drain. The wood screed floor remains dry. The screed members should be rot-resistant, i.e., redwood, cypress, cedar, or chemically treated pine. Neither of these options address structural concerns per se. The expressed intent is to control unwanted water.

7B.5.5 Conclusion

When basement wall failures occur in a vertical direction, underpinning and mudjacking are employed as discussed in Sections 7B.2 and 7B.4. To avoid extensive excavation, the repair work is often performed from inside the basement.

There are many other options to correct problems with basement/foundation walls; however, the foregoing should provide a sense of direction.

7B.6 SOIL STABILIZATION

7B.6.1 Introduction

Soil stabilization refers to a procedure for improving natural soil properties in order to provide more adequate resistance to erosion, loading capacity, water seepage, and other environmental forces. In foundation or geotechnical engineering, soil stabilization is divided into two sections: 1) mechanical stabilization, which improves the structure of the soil (and consequently the bearing capacity), usually by compaction, and 2) chemical stabilization, which improves the physical properties of the soil by adding or injecting a chemical agent such as sodium silicate, polyacrylamides, lime, fly ash, or bituminous emulsions. Generally, the chemical either reacts with the soil or provides an improved matrix that binds the soil.

In residential foundations, soil stabilization refers not only to improving the compressive strength or shear strength but also to increasing the resistance of the soil to dynamic changes(usually water-related) The latter tends to destroy both the soil's integrity and its structure. Generally, the former relates to stress applied to the soil by the foundation and the latter to the conditions imposed by the environment. Both are relative to soil characteristics.

Among the different mechanical stabilization techniques, such as preloading (to reduce future settlement), moisture control (to accelerate settlement), and compaction or densification (to improve bearing capacity and/or reduce settlement), compaction is generally the least expensive alternative for residential and commercial buildings. Detailed and specific information can be found in Section 6A.6.

7B.6.2 Compaction

Compaction may be accomplished by excavating the surface soil to a depth for residential buildings up to 4 ft (1.3 m) and for commercial buildings up to 6 ft (1.8 m), and then back filling in controlled layers and compacting the fill to 95% compaction. Often, the fill material is a replacement type such as some nonplastic (low plasticity index) soil. In the case of uniform soil (sand), the addition of a fine soil to improve the grain size distribution is advised. The standard compaction tests utilized to evaluate these processes include one of the following:

1. ATSM D698-70. 5.5 lb. hammer, 12 in drop, 1/30 ft^3 mold; three layers of soil at 25 blows per layer may be used.
2. ATSM D-1557-70. 10 lb. hammer, 18 in drop, 1/30 ft^3 mold; five layers at 25 blows per layer may be used.

Specific details of compaction tests, equipment, and quality control are discussed in Section 6A.

The undrained shear strength of a soil acceptable for a housing site should not be less than 800 lb/ft^2 (38 kPa). In a nonexpansive fill with less than 600 lb/ft^2 (29 kPa) undrained shear strength, compaction, preloading or grout injection are methods beneficial for improving the soil for light residential construction. For heavier foundation loads, piers or other forms of structural enhancement might be required. This might encompass a more sophisticated foundation design, soil improvement, or both. A discussion of specific soils amenable to mechanical improvement follows.

7B.6.3 Granular Soil

Granular soils are those composed of particles larger than 0.0075 mm (No. 200 sieve). After proper compaction, most granular soils are modified to give them volume stability and improved frictional resistance. Still, they often retain high permeability. In order to offset this property the soils might be blended with either a granular material to provide a well-graded soil or a choesive material to provide bonding or cementation. The latter is intended to provide cohesion under both moist and dry conditions. Silty clay constituents (or other cementitious materials) can also decrease the danger of other instability under either dry or, particularly, wet conditions. Laboratory tests are performed to determine the optimum conditions of compaction or soil modification.

In general, a sandy soil, particularly after stabilization, has a high bearing capacity, but foundations should be placed at a sufficient depth so the soil beneath the loaded member is confined. Foundations in stabilized sand may consist of spreadfootings, mats, piles, or piers, depending principally on the soil density and thickness, cost of soil modification, and imposed loads. In sand deposits (without compaction) spreadfootings are used if the deposit is sufficiently dense to support the loads without excessive settlement. Piers in loose sand deposits should be drilled to firm underlying strata. Skin friction can be considered in the design (or load) criteria for sand or granular soil. The foundation should not be located on sand deposits where relative density is less than 60% or a density of 90% of the maximum cannot be attained in the soil laboratory. An exception would be where the loose material is completely penetrated (usually by driven piles) as noted above.

7B.6.4 Foundation on Loess

Loess is a fine-grained soil formed by the deposit of wind-borne (aeolian) particles. This soil covers 17% of the United States (refer to Figure 7B.6.1). Depths of loess deposits range from 1 to 50 m (3.3 to 165 ft) and depths from 2 to 3 m (6.6 to 9.9 ft) are common. Loess has a specific gravity from about 2.6 to 2.8 and in situ dry density from about 66 to 104 lb/ft^3 (1057 to 1666 kg/m^3) with 90% particle size passing a No. 200 sieve. The plastic limits range from 10 to 30%. Standard compaction tests produce dry densities of 100 to 110 lb/ft^3 (1602 to 1762 kg/m^3) at optimum moisture content of 12 to 20%. As a foundation soil, a loess soil with a density greater than 90 lb/ft^3 (1442 kg/m^3) will often exhibit only limited settlement. The problem with loess is the changing of bearing capacity with saturation. Upon saturation, soil bearing capacity can drop to 90% or less than that of the dry loess. Loess is silt, cemented by calcareous materials. The addition of water destroys the cement bonds. (Eroded loess is commonly referred to as a silt deposit.) Loess below the permanent water table is, as one would guess, relatively stable because the water content is constant. Compacted loess can be a satisfactory foundation material for mats and spreadfootings if the density is more than 1.6 g/cm^3 or the bulk density is higher than about 100 lb/ft^3 (1600 kg/m^3).

Loess can be stabilized by using lime, lime fly ash, or cement, each followed by compaction. Piers are commonly suggested if in-place specific gravity of the loess is under 1.44 g/cm^3 (90 lb/ft^3 or 1440 kg/m^3). Piles should be driven, or piers drilled, through the loess into the underlying soil layer unless the loess terminates below the water table. Again, loess is often stable within or below the water table.

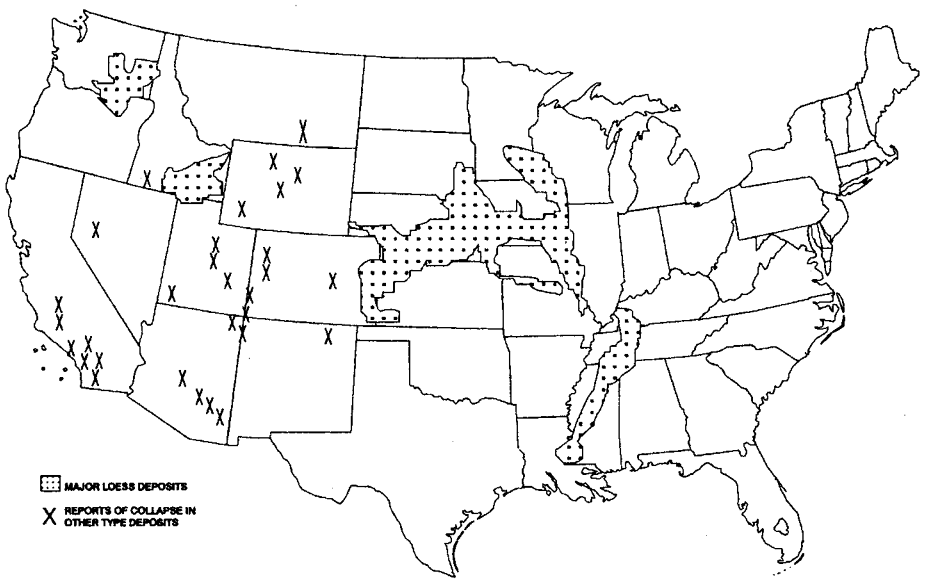


FIGURE 7B.6.1. Location of major loess deposits in the United States. (Adapted from Dudley, 1970; used with permission of ASCE.)

7B.6.5 Foundations on Sanitary Landfill Sites

"Sanitary landfill" is a euphemism for a garbage dump. Within urban areas, it often becomes necessary to develop a former sanitary landfill for construction. In most instances, the opportunity for soil improvement by normal compaction techniques is denied. Often, the preparation of a site for construction would require extensive grouting to fill voids and consolidate the soil. The alternative would be a foundation design that penetrated the fill section into competent material.

Landfill usage for one- or two-story residential buildings, apartments, office buildings, or other light construction, can be acceptable if the site is either adequately compacted or stabilized by grout injections or modified by the addition of lime or cement. This would assume the required bearing capacity of the soil (including the factor of safety) to be within the range of 0.5 to 1.0 tons/ft^2 (4.88 to 9.76×10^3 kg/m^2). In this way, the use of continuous foundations may provide adequate bridging capacity over local soft spots or cavities. Otherwise, piers should be extended to a firm layer underlying the landfill.

7B.6.6 Stabilizing Permeable Soils

Generally, the concerns with noncohesive, permeable soils involve measures to control or prevent sloughing, improve bearing strengths, reduce creep or lateral shifting, control water flow, etc. To best provide this function, stabilizing chemicals that develop a cementitious matrix are preferred. These additives include such materials as cement slurry, fly ash or pozzolanic earth in lime or cement slurries, sodium silicate (water glass) mixed with a strong acid, or methyl methacrylate polymerized by a peroxide catalyst.[17] The basic nature of the individual soil particle is unchanged; the particles are merely cemented together by the cementitious material filling the void (or pore) space.

As a rule, the stabilizing materials are introduced into the soil, through some variation of pressure injection, often to depths of 10 to 20 ft (3 to 6 m). Injection pipes are generally mechanically driven or washed down by water to total depth. Once grout injection commences, the pipe is slowly withdrawn. In other words, injection proceeds from the bottom toward the surface and generally continues at each level until either refusal or some predetermined volume is placed. Varying from one type of material to another usually involves little more than changing the mixer and/or pump. For more information on this subject, refer to Section 7B.3.

7B.6.6.1 Pressure Grouting

Pressure grouting is also often used to improve the bearing capacity of soils, whether impermeable or permeable. The key is to either compress the soil material to a level above the anticipated load or create a soil matrix possessing the desired bearing capacity. For example, if a compressible organic/inorganic fill could be grouted to an actual pressure of 100 lb/in^2 (7 kg/cm^2), the soil theoretically could support a load of 14,400 lb/ft^2 (70,000 kg/m^2), not taking into account any factors. (Actual pressure implies the true compressive pressure on the soil matrix, and not gauge pressure at the surface.) (Also see Sections 6B and 7B.3.)

7B.6.7 Stabilizing Impermeable Soils

For purposes of our discussion, the impermeable soils are generally cohesive with appreciable clay content. As a rule, the clay constituent will be one of expansive nature. The problems generated by a clay are influenced directly by variations in available moisture, the result of which is either shrinking or swelling. This volatile nature causes serious concern regarding the design, construction, and stability of foundations.

For example, a typical Eagle Ford soil (Dallas, Texas) with a PI of about 42, will exhibit a confined swell pressure of about 9000 lb/ft^2 (44,000 kg/m^2) when the moisture content is increased from 23 to 26%. In this example, the problem clay constituent is montmorillonite, which is present at up to 50% of the total solids volume.[102,103] Considering that the preponderant weight of a residential or light commercial structure is carried by the perimeter beam and that load is appreciably less than 1000 lb/ft^2 (4880 kg/m^2) (single-story construction), it becomes obvious that structural instability is imminent. What must eventually happen if the soils' upward thrust is 9000 lb/ft^2 (44,000 kg/m^2) and the maximum structural resistance is less than 1000 lb/ft^2)? The building will rise! [The interior floor area often represents loads as low as 50 to 100 lb/ft^2 (240 to 490 kg/m^2).]

Because of the difference in structural resistance as well as the heterogeneous nature of the soil, the uplift or heave is seldom, if ever, uniform. The secret is to deny the soil the 3% change in moisture or alter the properties of the clay constituent to the extent that influences of differential water are neutralized. The former was discussed in Section 7A; the latter can be accomplished by treating the problem soil with certain chemical agents. The stabilization procedure depends to some extent on one's comprehension of the nature of the specific clay constituents. When the subject soil is basically an expansive clay, compaction alone is most often inadequate to prepare the site for foundation support. It is desirable to alter the soil's behavior by either the use of chemicals or pressure grouting or, occasionally, perhaps a combination of both. Overconsolidation of shallow or surface soils should be avoided.

7B.6.7.1 Chemical Stabilization

The soil upon which a foundation is supported influences or dictates the structural design and ultimate stability of the constructed facility. Earlier chapters have dealt with the pertinent physical properties inherent to and desirable within a bearing soil. Some discussion has been devoted to problem soil components such as the expansive clays. Criteria for overcoming the clay problems through design and maintenance of the foundation have been discussed. This section will address other options: 1) impart beneficial properties to the otherwise problem soil, or 2) alter the offending clay constituent to reduce or eliminate the volatile potential. Chemical stabilization represents a classic approach to this problem and can be separated into two categories: 1) permeable soils (which generally include noncohesive materials such as sand, gravel, organics, and occasionally silts) have been discussed in preceeding paragraphs, and 2) the nonpermeable soils, which generally are cohesive in nature and contain the clays (e.g., montmorillonite, attapulgite, chlorite, illite, and kaolinite). These are also referred to as expansive soils (except kaolinite) and often need to be stabilized to control shrink–swell.

Actually, "stabilization" can be a little misleading. Most chemicals are designed to abate swell. In order to eliminate settlement, transpiration would need to be eliminated, which translates to no live plant roots. This option is not looked upon favorably with most home owners. Some products offer selective side benefits in the form(s) of: increased permeability, increased bearing strength, and, perhaps, some decrease in shrink. Still, the principal intent is to reduce swell.

One of the less expensive stabilization methods would be to maintain the in situ soil moisture at a constant level. This can sometimes be easier said than done. Several approaches have been proposed in the literature such as 1) sophisticated watering systems,[15–17] and 2) moisture barriers.[15–17,26,79] Proper watering has shown the most potential. The other choice is to chemically modify the clay to control the volatile nature of the soil.

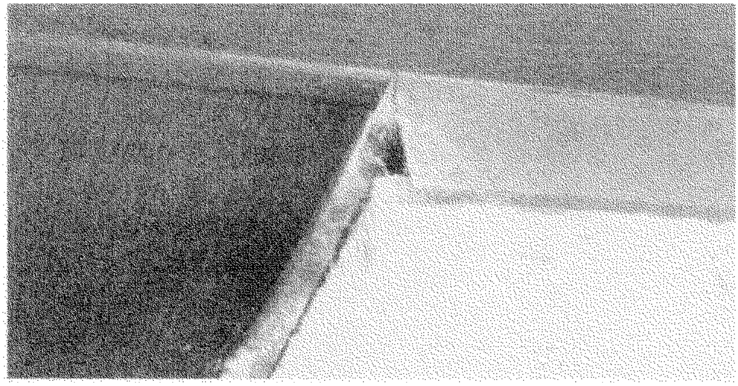
7B.6.7.2 Inorganic Chemicals

Stabilization by inorganic chemicals has been rather widely used. The principal mechanism for this reaction is that of cation exchange. The increased positive charges hold the clay platlets closer together, inhibiting swell. Cementitious benefits are sometimes realized.[15–17] Use of lime, $Ca(OH)_2$, has been a common occurrence for over 30 years, particularly for highway construction. The lime is intimately mixed with the base and/or subbase soils (tilled into the matrix), watered down, and compacted. The calcium cation exchanges with clay constituents and may produce some pozzolanic reactions with silicas. Both actions afford stabilization to the soil.[15–16] Tilling has also been used successfully to some extent in new residential construction. However, the process has not met with general acceptance for remedial applications. This is largely due to cost, problems in obtaining an adequate mix of the lime into the soil matrix, questionable results, and construction delay due to the wet site. A modification of the lime application was introduced in the 1960s when a lime slurry [$Ca(OH)_2$ in water] was pressure injected into the soil (LSPI). In a further effort to facilitate penetration of the slurry into the soil, a surfactant (organic) was frequently added. Pressure lime injection has experienced moderate success in new construction. The principal drawback still focuses on the very low solubility of lime in water and the vertical impermeability of the expansive soils. This technique was ultimately tried on remedial projects with much less success.[3,15–17,26,79] Figure 7B.6.2 documents a monumental failure for LSPI. This depicts a foundation severely damaged from movement induced by pressure lime injection. Following the initial treatment, no appreciable amount of leveling was noted (as would be expected); however, it was assumed that stabilization had been accomplished. Complete cosmetic repairs were performed. Two years later, the observations shown in Figure 7B.6.2 were noted. The photos speak for themselves. The elevation taken (Figure 7B.6.2d) suggest a "new" differential movement in the range of 5 in (12.5 cm). Note also the washboard nature of the slab surface. This problem was ultimately remedied by underpinning and mudjacking the slab foundation. In other applications, the stabilizing chemical might involve a mixture of potassium chloride (KCl), an organic surface active agent (surfactant), and perhaps sulfuric acid (H_2SO_4). Aside from the obvious hazardous aspects of handling, the reaction of sulfuric acid with lime, already present in the soil, offers yet another serious concern. Lime and sulfates react within the wet clay soils to produce ettringite, a calcium, aluminum, sulfate hydrate ($3CaO\text{-}Al_2O_3\text{-}3CaSO_4\text{-}32$

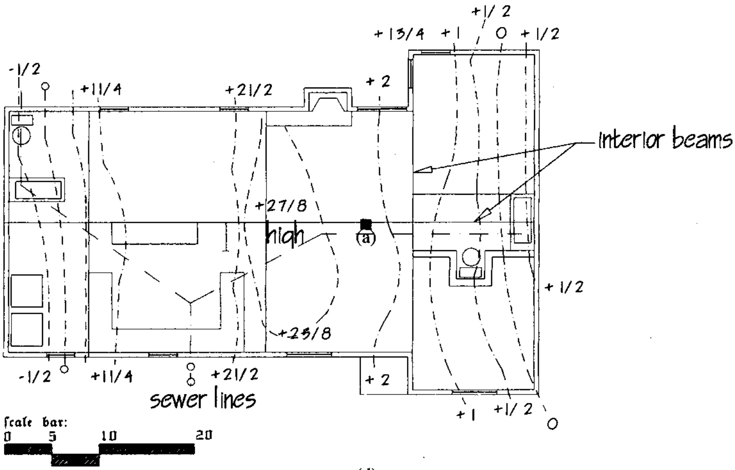
(a)

(b)

FIGURE 7B.6.2. Failure due to pressure lime injection. (a) Sheetrock cracks in wall and ceiling; (b) separation in brick mortar with lateral displacement of brick.



(a)



(d)

FIGURE 7B.6.2. (*continued*). Failure due to pressure lime injection. (c) Separation in brick frieze; (d) realtive elevation survey.

H_2O). This product has an unpredictable and uncontrollable swell.[3,82,85,106] (Along this same train of thought, the use of lime in clay soils containing sulphate has resulted in the same problems.) The use of KCl plus surfactant would probably prove to be a better, certainly safer, product without H_2SO_4. (Several States, California in particular are reporting problems with sulfate deterioration of concrete. This problem was a topic of discussion at the slab-on-ground committee conference in Dallas, TX, Sept. 1999. At that time, the principal concern was natural sulfates contained within the soils. The use of chemical soil stabilizers that contain sulfates would grossly acerbate the problem).

7B.6.7.3 Organic Chemicals

Over the years, several organic-based products have been used to stabilize the clays with varying degrees of success. One such product, Soil Sta, is the lone product for which reliable data can be found in the literature. Essentially, this product is a mixture of a poly-quatenaryamine and surfactant in water. The product is neither toxic nor hazardous. In over 6000 applications, not one single failure to abate intolerable swell has been recorded. In most tests, the soil swell potential was reduced to less than 1% at moisture contents of 17% or higher (PL 30%). Refer to Figure 7.B6.3 The product has been tested to increase the soil's permeability by a factor of eight, increase the soil resistance to shear two-fold, and reduce soil shrinkage by ranges of 11 to 50%.[15–17] The organic chemicals have an extremely high solubility in water, acceptable permeation into soil matrix, reasonable cost, and a most effective performance.

One concern, however, is choosing the proper condition under which the use of the chemical(s) is cost effective. When the primary concern is to abate swell, the use of Soil Sta is probably not warranted if the soils' natural moisture content (W%) is in the range of the PL. At this moisture level, a high percentage of the soils' swell potential has already been realized.[15–17] In other words, without the introduction of chemical stabilizer, additional water is not likely to cause significant soil swell.

On the other hand, if the soil's in situ moisture is appreciably lower than the PL, Soil Sta could be an effective choice. The greater the range between W% and PL, the more effective will be the performance. The product is most effective on the montmorillonite clays.

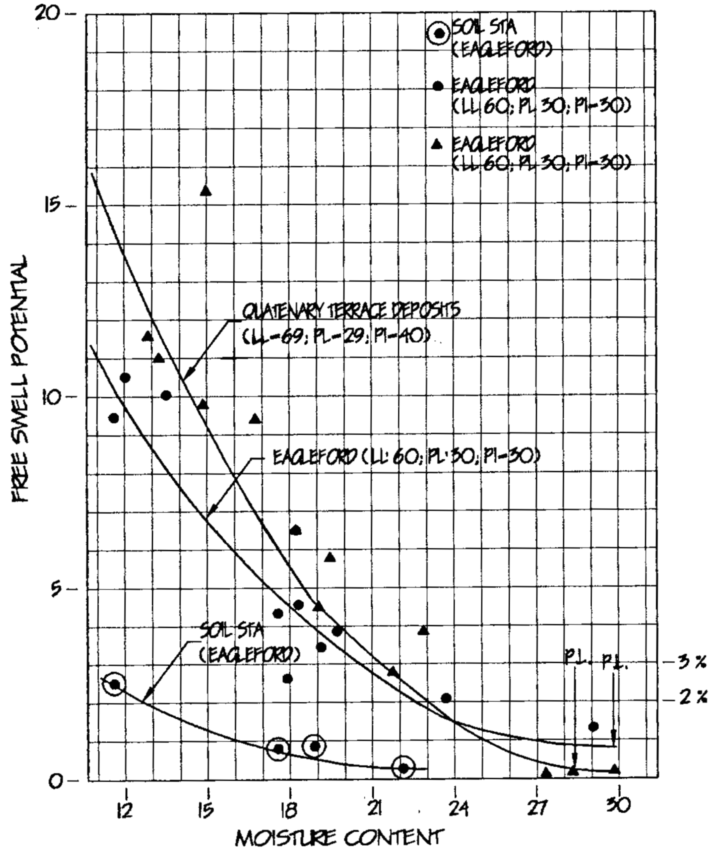


FIGURE 7B.6.3. Free swell versus moisture content.

7B.6.8 Clay Mineralogy

Basically, the surface clay minerals are composed of various hydrated oxides of silicon, aluminum, iron, and, to a lesser extent, potassium, sodium, calcium, and magnesium. Since clays are produced from the weathering of certain rocks, the particular origin determines the nature and properties of the clay. Chemical elements present in a clay are aligned or combined in a specific geometric pattern referred to as structural or crystalline lattice, which is generally sheetlike in appearance. This structure, coupled with ionic substitution, accounts principally for both the various clay classifications as well as their specific physical/chemical behavior.

By virtue of a loose crystalline structure, most clays exhibit the properties of moisture absorption and ion exchange. Among the more common clays, with the tendency to swell in decreasing order, are montmorillonite, illite, attapulgite, chlorite, and kaolinite. Figure 7B.6.4 shows the areas of general and local abundance of high-clay, expansive soils. The darker areas indicate those states suffering most seriously from expansive soil problems.

The data (provided by K. A. Godfrey, *Civil Engineering*, October 1978) indicate that nine states have extensive, highly active soils and eight others have sufficient distribution and content to be considered serious. An additional 10 to 12 states have problems that are generally viewed as scattered or relatively limited. As a rule, the 17 "problem" states have soil containing montmorillonite, which is, of course, the most expansive clay. The 10 to 12 states with so-called limited problems (represented by the lighter coloring) generally have soils that contain clays of lesser volatility, such as illite and/or attapulgite or montmorillonite in lesser abundance.

A specific clay may adsorb water to varying degrees—from a single layer to six or more layers—depending on its structural lattice, presence of exchange ions, temperature, environment, and so on. The moisture absorbed may be described as one of three basic forms: interstitial or pore water, surface adsorbed water, or crystalline interlayer water. This combined moisture accounts for the differential movement (e. g., shrinking or swelling) problems encountered with soils. In order to control soil movement, each of these forms of moisture must be controlled and stabilized.

The first two forms, interstitial (or pore water) and surface adsorbed water, are generally accept-

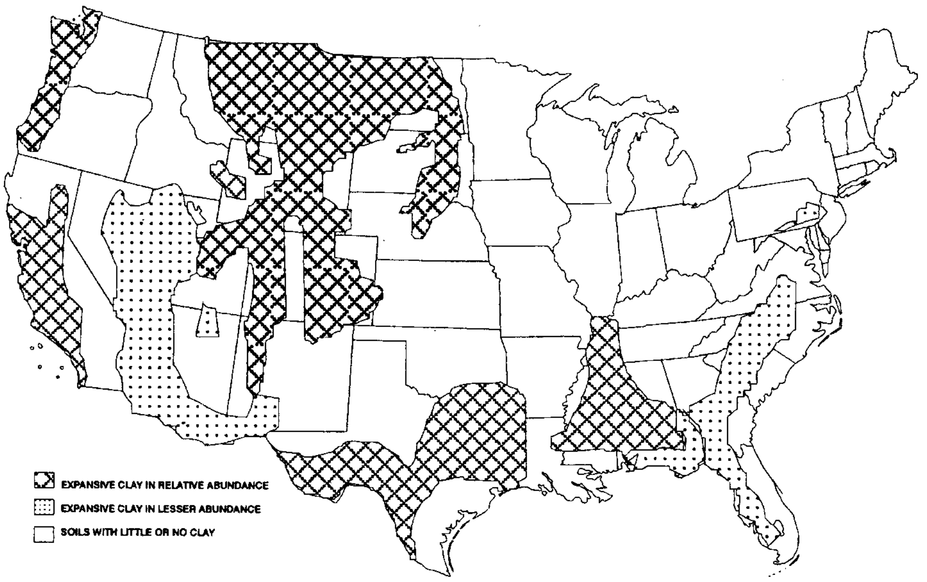


FIGURE 7B.6.4. Distribution of expansive soils in the United States.[57]

ed as capillary moisture. Both occur within the soil mass external to individual soil grains. The interstitial or pore water is held by interfacial tension and the surface adsorbed water by molecular attraction between the clay particle and the dipolar water molecule. Variations in this combined moisture are believed to account for the principal volume change potential of the soil. Refer also to the Introduction to this volume.

Soils can take on or lose moisture. Within limits, this moisture exchange involves pore or capillary water, sometimes referred to as free water. (It is recognized that capillary water can be transferred by most clays to interlayer water, and vice versa. However, the interlayer water is normally more strongly held and, accordingly, most stable. This will be discussed at length in the following paragraphs.)

In a virgin soil, the moisture capacity is frequently at equilibrium, even though the water content may be well below saturation. Any act that disturbs this equilibrium can cause gross changes in the moisture affinity of the clay and result in either swelling or shrinking. Construction, excavation, and/or unusual seasonal conditions are examples of acts that can alter this equilibrium. As a rule, environmental or normal seasonal changes in soil moisture content are confined very closely to the ground surface. That being the case, it would appear that for on-grade construction, it should be sufficient to control the soil moisture only to this depth.

At this point, it should be emphasized that, for capillary water to exist, the forces of interfacial tension and/or molecular attraction must be present. Without these forces, the water would coalesce and flow, under the force of gravity, to the phreatic surface (top boundary of water table). The absence of these forces, if permanent, could fix the capillary moisture capacity of the soil and aid significantly in the control of soil movement. Control or elimination of soil moisture change is the basis for chemical soil stabilization.

Interlayer moisture is the water situated within the crystalline layers of the clay. The amount of this water that can be accommodated by a particular clay depends on three primary factors: the crystalline spacing, the chemical elements present in the clay crystalline structure, and the presence of exchange ions. As an example, bentonite (sodium montmorillonite) will swell approximately 13 times its original volume when saturated with fresh water. If the same clay is added to water containing sodium chloride, the expansion is reduced to about three-fold. If the bentonite clay is added to a calcium hydroxide solution, the expansion is suppressed even further, to less than two-fold. This reduction in swelling is produced principally by ion exchange within the crystalline lattice of the clay. The sorbed sodium ions (Na^+) or calcium ions (Ca^{2+}) limit the space available to the water and cause the clay lattice to collapse and further decrease the water capacity.

As a rule (and as indicated by this example), the divalent ions such as Ca^{2+} produce a greater collapse of the lattice than the monovalent ions such as Na^+. An exception to the preceding rule may be found with the potassium ion (K^+) and the hydrogen ion (H^+). The potassium ion, because of its atomic size, is believed to fit almost exactly within the cavity in the oxygen layer. Consequently, the structural layers of the clay are held more closely and more firmly together. As a result, the (K^+) becomes abnormally difficult to replace by other exchange ions. The hydrogen ion, for the most part, behaves like a divalent or trivalent ion, probably through its relatively high bonding energy. It follows that, in most cases, the presence of H^+ interferes with the cation exchange capacity of most clays. This has been verified by several authorities. R. G. Orcutt et al. indicate that sorption of Ca^{2+} by halloysite clay is increased by a factor of nine as the pH (OH^- concentration) is increased from 2 to 7.[81]

Although these data are limited and qualitative, they are sufficient to establish a trend. R. E. Grim indicates that this trend would be expected to continue to a pH range of 10 or higher.[45,46] [The cation exchange at high pH, particularly with Ca^{++}, holds significant practical importance.] This is the basis for stabilization of expansive soils with lime [Ca(OH)2]. The pH is defined as the available H^+ ion concentration. A low pH (below 7) indicates acidity, 7 is neutral, and above 7 is basic. Cement or lime stabilization of roadbeds represents one condition in which clays are subjected to Ca^{2+} at high pH. It should be recognized that under any conditions the ion-exchange capacity of a clay decreases as the exchanged-ion concentration within the clay increases. Attendant on this, moisture-sorbtion capacity (swelling) decreases accordingly.

The foregoing discussion has referred to changes in potential volumetric expansion brought about by induced cation exchange. In nature, various degrees of exchange preexist, giving rise to widely variant soil behavior even among soils containing the same type and amount of clay. For example, soils containing Na^+-substitute montmorillonite will be more volatile (expansive) than will soils containing montmorillonite with equivalent substitution of $Ca^{2=}$ or Fe^{3+}. This is true because Na^+ is more readily replaced by water and the absorption and/or adsorption of the water causes swell. Also, the high valence tends to hold the clay platelets in closer contact, inhibiting the intrusion of water.

To this point, the discussion has focused on inorganic cation exchange. However, published data indicate that organic ion adsorption might have even more practical importance to construction problems.[35] The exchange mechanism for organic ions is basically identical to that discussed above, the primary difference being that, in all probability, more organic sorption occurs on the surface of the clays than in the interlayers, and, once attached, is more difficult to exchange. Gieseking[43] reports that montmorillonite clays lost or reduced their tendency to swell in water when treated with several selected organic cations. The surface adsorbed water (or double diffuse layer) that surrounds the clay platelets can be removed or reduced by certain organic chemicals. When this layer shrinks, the clay particles tend to pull closer together (flocculate) and create macropores or shrinkage cracks (intrinsic fractures). The effect of this is to increase the permeability of the expansive soil.[45]

This action should be helpful to reduce ponding, reduce run-off, and facilitate chemical penetration for stabilization. The extent of these benefits depends on the performance of the specific chemical product. Specific chemical qualities tend to help promote the stabilization of expansive clays. Among these are: high pH, high $[OH]^-$ substitution, high molecular size, polarity, high valence (catonic), low ionic radius, and highly polar vehicle. Examples of organic chemicals that possess a combination of these features include: polyvinylalcohols, polyglycolethers, polyamines, polyquarternaryamines, polyacrylamides, pyridine, collidine, and certain salts of each. Since no single one of the above organic chemicals possesses all the desired qualities, they are generally blended to enhance the overall performance. For example, the desired pH can be attained by addition of lime $[Ca(OH)^2]$, hydrochloric acid (HCl), or acetic acid (C2H3OOH); the polar vehicle is generally satisfied by dilution with water; high molecular size can be accomplished by polymerization; surfactants can be utilized to improve penetration of the chemical through the soil; and inorganic cations can be suppled to provide additional base exchange. (For more detailed information concerning the chemical reactions of base exchange in montmorillonite, refer to Sections 6.6.3 and 6.6.4. *Foundation Behavior and Repair*.[16]) Generally, the organic chemicals can be formulated to be far superior to lime with respect to clay stabilization. Organic chemicals can be selected that are soluble in water for easy penetration into the soil. Chemical characteristics can be more finitely controlled and the stabilization process can be more nearly permanent. About the only advantages lime has over specific organics, at present, are lower treatment cost, more widespread usage (general knowledge), and greater availability.

The point will be made in later discussions that foundation repair generally is intended to raise the lowermost areas of a distressed structure to produce a more nearly level appearance. The repair could be expected to be permanent only if procedures were implemented to control soil moisture variations. This is true because nothing within the *usual* repair process will alter the existing conditions inherent in an expansive soil. Alternatively, chemical stabilization can alter the soil behavior by eliminating or controlling the expansive tendencies of the clay constituents when subjected to soil moisture variations. If this reaction is, in fact, achieved, the foundation will remain stable, even under adverse ambient conditions.

Several organic-based products are currently available to the industry. One such product, Soil Sta, is discussed in Sections 7B.6.9 and 7B.6.12. This particular product was selected principally because of the availability of the reliable data and its documented effectiveness.

7B.6.8.1 Properties for a Clay Stabilizing Chemical

A superior chemical stabilizing agent must be both economical and effective. However, the definitions of these terms can be quite arbitrary. As a start, the economical aspect is assumed to be at a cost somewhat competitive with that for conventional lime. The effective aspect is more elusive. The chemical should be:

1. Effective in reducing swell potential of clay
2. Reasonably competitive with lime in cost, but more readily dispersed into the soil. (Lime is sparsely soluble in water and therefore difficult to use in expansive soils where cutting and tilling is inappropriate.)
3. Compatible with other beneficial soil properties
4. Free from deleterious side effects such as a corrosive action on steel or copper, herbicidal tendencies, unpleasant smell, and hazards to health or the environment
5. Easy to apply with few, if any, handling problems for the applicators or equipment
6. Permanent

At first, it might seem that the chemical should actually dehydrate clay or, in field terms, "shrink the swollen soil." The problem lies in the fact that such dehydration is most often unpredictable and nonuniform. It seems that a simpler approach would be to treat the clay to prevent any material change in the water content within the clay structure. Soil Sta was formulated to meet all the noted criteria. The mechanism by which this occurs is to both replace readily exchangeable hydrophilic cations (such as Na^+) and adsorb on the exposed cation exchange sites to repel invading water.

7B.6.8.2 Chemistry of Cation Exchange

The chemistry of cation exchange and moisture capacity within a particular clay is neither exact nor predictable. A given clay under seemingly identical circumstances will often indicate variations in cation exchange as well as the extent to which a particular cation is exchanged. The specifics of this exchange capacity dictate the water affinity and bonding to the clay structure. Refer to Section 6A.6 for details on this topic.

In addition to the foregoing, many organic chemicals tend to shrink the double diffuse layer that surrounds the clay particles, causing the clay particles flocculate and the soil skeleton to shrink. The net result is the formation of cracks (referred to as syneresis cracks). The combination of these effects coupled with attendant desiccation increases the permeability of the clay.[38] The exposure of greater surface area may further facilitate the base exchange of certain organic molecules.[11,47,59] The chemical should then prevent swell upon reintroduction of water.

7B.6.9 Development of a Soil Stabilizing Chemical

For some time, different groups have tested various chemicals as stabilizing agents to prevent the extensive swell of specific clays, in particular montmorillonite. These studies have involved the petroleum industry as well as the construction industry. As a result, several materials exhibiting varying potential have evolved. However, the general emphasis has been on new construction. Considering all factors, hydrated lime [$Ca(OH_2)$], has been difficult to displace in types of applications in which a controlled, intimate mix with the clay/soil was feasible.

In remedial applications, such as stabilizing the soil beneath an existing structure, lime has its inherent shortcomings: it is difficult to introduce into the soil matrix with any degree of uniformity, penetration, and saturation. This stems largely from the facts that 1) lime is sparsely soluble in water and 2) the clay/soil needing stabilization is both impermeable and heterogeneous. A comprehensive state-of-the-art report on lime stabilization can be found in Dr. J. R. Blacklock's publications, the latest of which is referenced.[5] However, the use of hydrated lime to stabilize montmorillonite clays can also create detrimental side reactions. In a study presented by Berry Grubbe,[48] lime stabilization of naturally expansive soils (Austin Chalk and Eagle Ford) resulted in significant heave to a pavement section. The heave reportedly was caused by the chemical reaction between lime and sulfates in the soil to produce ettringite. Refer to Section 7.7.2. Obviously, although this report addresses distress to pavements, the soil swell (heave) problem would be the same in other applications of lime stabilization. Further, Tom Petry and D. N. Little describe soil heave brought about by sulfate introduction into clay soils containing lime.[85] In large part because of the foregoing, the successful use of lime in soil stabilization has not been documented for remedial applications.

As early as 1965, certain other surface-active organic chemicals were evaluated and utilized with some degree of success. One very successful chemical utilized in the late 1960s and early 1970s was unquestionably successful in stabilizing the swell potential of montmorillonite clay. The chemical was relatively inexpensive and easily introduced into the soil. However, the product maintained a "nearly permanent" offensive aroma that chemists were never able to mask. Generally, this product was a halide salt of the pyridine-collidine-pyrillidene family.

In the late 1970s, the quest began to focus more on the potential use of polyamines, polyethanol glycolethers, polyacrylamides, etc., generally blended, and containing surface-active agents to enhance soil penetration.[104] It was found that certain combinations of chemicals seemed to be synergistic in behavior (the combined product produced superior results to those noted for any of its constituents). In the mid 1980s, one such product was Soil Sta.*

Soil Sta is basically a mixture of surfactant, buffer, inorganic cation source, and polyquaternaryamine in a polar vehicle. By virtue of its chemical nature, Soil Sta would be expected to have a lesser influence on kaolinite or illite than on the more expansive clays such as montmorillonite. Prior research has also indicated that soils exhibiting liquid limit (LL) less than 35 or plasticity index (PI) less than 23 (montmorillonite. content less than about 10% by weight) would not swell appreciably.[6] Hence, the soils utilized in the laboratory tests and field applications contained montmorillonite as a soil constituent above 10%.

Soil Sta was first subjected to laboratory evaluation in 1982, and field testing commenced in mid 1983. The laboratory tests indicated that Soil Sta:

1. Reduced the free swell potential of montmorillonite clay (Figure 7B.6.3)
2. Appeared stable in repeated weather cycles (a simulated period of 50 years)
3. Increased shear strengths in some soils by two-fold
4. Increased soil permeabilities up to 40-fold
5. Reduced soil shrinkage by amounts varying from 11 to 50%[17,70,84]

By 1991, Soil Sta had been subjected to literally thousands of field applications with few, if any, failures. That is, less than 1% of the foundations treated with the chemical experienced recurrent movement. With those that did, there was a serious question as to the cause.

7B.6.9.1 Pressure Injection

In special cases in the United States and for most applications within the United Kingdom, chemical injection is performed through a specially designed system (Figure 7B.6.5). In the system utilized, the Soil Sta is injected under pressure to some depth, usually 4 to 6 ft (1.2 to 1.8 m). Penetration of the stem is accomplished by pumping through the core and literally washing the tool down. It is difficult, if not impossible, to wash the stem down when the base course is rubble or coarse gravel. In this instance, a pilot hole through the fill material is required. This can be accomplished by using a paving breaker and steel point to penetrate the problem base. This done, normal placement of the stem can continue. Once positioned, the hand valves are switched to close the core and divert flow

*The product Soil Sta is proprietary to the author. This presentation is not intended to be commercial. In fact, Soil Sta is not marketed. Necessity suggests the focus on this particular product because similar laboratory and field data are not publicly available for any other stabilizer, except perhaps lime. Organic stabilizers function differently from lime, and no standardized testing procedures existed for the evaluation of these type products. The following discussions and data should prove beneficial to the others wishing to evaluate an organic chemical clay stabilizer. All descriptive data and information was supplied through the courtesy of Brown Foundation Repair and Consulting, Inc., Dr. Cecil Smith, Professor of Civil Engineering, Southern Methodist University, Dr. Tom Petry, Professor of Civil Engineering, University of Texas, Arlington, all of Dallas, Texas, and Dr. Malcom Reeves Soil Survey of England and Wales, London, England.

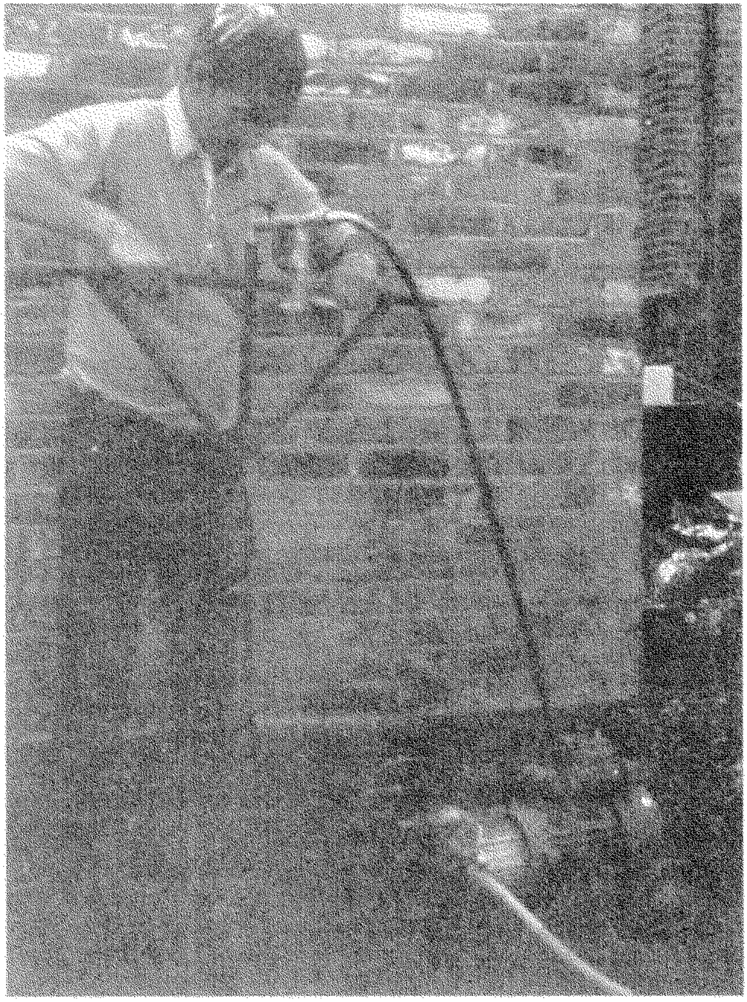


FIGURE 7B.6.5. Chemical injection through a specially designed stem.

into the annular space and out the injection ports (Figure 7B.6.6). The stem can be raised during pumping to cover the desired soil matrix section. Generally, the treatment volume is determined on the basis of ⅛ gal/ft^2 or 0.5 mL/cm^2. In some instances, a particular soil might tend to resist Soil Sta penetration. Both the rate of penetration and the volume of chemical placed can be enhanced by utilizing hydraulic pulsation (high pressure of short duration) during the injection phase. Alternatively, the stem can be equipped with a packer assembly to selectively isolate zones (Figure 7B.6.7). This equipment permits high injection pressures and also allows zone selectivity.

From a practical viewpoint, minimal concern should be given to the exact volume of chemical injected into a specific hole. The primary intent is to distribute the treatment volume reasonably uniformly over the area to be treated. Time (days, weeks, or months, depending on the specific site conditions) will produce a nearly equal distribution.

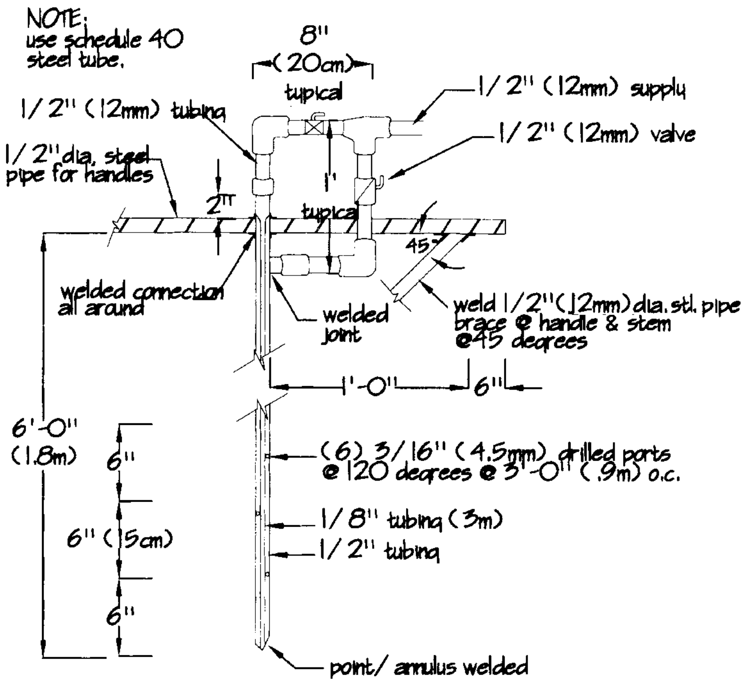


FIGURE 7B.6.6. Pressure injection stem.

A similar analysis would be true for depth of injection. Within a shallow depth [i.e., 6 ft (1.8 m) or less] the chemical will penetrate to the same approximate depth and interval almost independant of the position of the stinger. Shallow penetration is accompanied by problems of confining the permeation of the chemical into the matrix. Two changes could facilitate better chemical control: 1) the depth of penetration could be increased substantially, that is, 15 to 20 ft (4.5 to 6 m); 2) a pulsing injection technique could be used; and/or 3) packers or other positive seal methods could be used to isolate each zone to be injected. Even in the latter case, true zone penetration may not occur if the placement pressure or specific soil characteristics favor communication between zones. To illustrate the point, no matter what precautions might be taken, the normal heterogeneous and fractured nature of the soil would tend to preclude exact placement of any specificed volume.

At a pressure differential of about 3.5 lb/in^2 (24 kPa), the system illustrated in Figure 7B.6.6 would theoretically place about 12 gal/min (45 L/min), neglecting line friction. Hence, 10 s would be required to place 2 gal (7.6 L) of chemical. [This volume equates to ⅛ gal/ft^2 on a 4 ft (1.2 m) spacing pattern.] By timing the injection period and maintaining a reasonably constant supply pressure, an acceptably uniform treatment spread would result. Carelessness in either timing or pressure would not be disastrous, so long as it was not blatant. [In field practice, a pressure differential of about 60 lb/in^2 (414 kPa) delivered 2 gal (7.6 L) of chemical in 30 s through the stinger and approximately 60 linear feet (18.3 m) of ½ in ID (1.27 cm) hose.] The following equations can be used to estimate velocities and pressure differentials.

The annular velocity is

$$V_a = Q/A = 1.84Q \text{ ft/s} \qquad (7B.6.1)$$

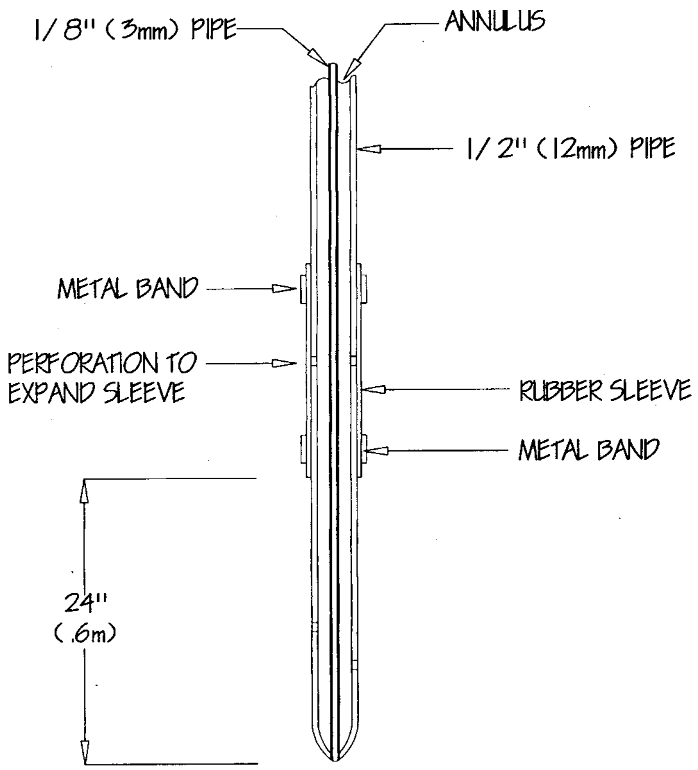


FIGURE 7B.6.7. Injection stinger with pack-off

where annular area $A = 0.175$ in^2 and Q is in gallons per minute. The port velocity is

$$V_p = 4Q/D_p^2\, nC_D \text{ ft/s} \tag{7B.6.2}$$

where $D_p = 3/16$ in, the number of ports $n = 6$, and Q is in gallons per minute.

Equation (7B.6.2) reduces to

$$V_p = Q/A_p C_D$$

where $A_p = D_p{}^2 n/4$. The orifice discharge coefficient C_D can be assumed to be 0.8.[20] The pressure differential is

$$P = p_f (V_p^2 - V_a^2)/149 \text{ lb/in}^2 \tag{7B.6.3}$$

where P_f is the specific gravity of the fluid (water = 1.0). The force developed from hydraulic pressure is

$$F = PA \tag{7B.6.4}$$

where F = force, lb$_f$ or kg; P = pressure, lb/in^2 or kg/cm^2; and A = area, in^2 or cm^2.

Equation (7B.6.4) is used to illustrate the factors creating a lifting force. Generally, pressure injection of chemical soil stabilizers does not involve lifting, as mudjacking or pressure grouting would. In fact, as a rule, chemical injection pressure could then be estimated from the rearrangement of Equation (7.4) as follows:

$$P = F/A$$

7B.6.10 Water Barrier

The use of chemical soil stabilizers (Soil Sta in particular) has focues renewed interest on the use of water barriers.[18] In areas where the in situ soil moisture is relatively high (nearing or exceeding the soil's plastic limit), a combined technique utilizing Soil Sta to control soil swell and a moisture barrier to prevent soil moisture loss (shrinkage) appears to have some merit. The use of Soil Sta in a relatively wet soil was predicated on the possibility that at some future point the soil moisture might be reduced substantially for a period of time and then increased back to or near the original level. This approach is being considered in the United Kingdom. In this instance, the slit trench [approximately 3 in (7.6 cm) wide by 60 in (1.5 m) deep, typically] is dug as near to the foundation perimeter as conditions permit and filled with concrete. This creates the moisture barrier (see also Figure 7C.1 and Section 7C.3.2). Soil Sta is injected according to the selected procedure as described in preceding paragraphs. The barrier is intended to prevent the peripheral loss of soil moisture due to either evaporation or transpiration. (Soil Sta has minimal effect on soil moisture loss to transpiration. Otherwise, the chemical would be detrimental to vegetation.) Obviously, the foregoing procedure is designed and intended to stabilize the soil moisture (and foundation) "as is" with negligible, if any, leveling. Where leveling is desired, or necessary, conventional methods are employed.

7B.6.11 Irrigation

An irrigation system similar to that depicted in Figures 6C.1 and 6C.2[17] could also overcome any peripheral loss of moisture. This system simply replaces any moisture otherwise lost from the soil by either evaporation or transpiration. The key to the effectiveness of this approach lies principally within the metering and monitoring equipment. The moisture returned by the system should be carefully controlled to replace water lost but at the same time maintain a constant soil moisture. Special care should be exercised not to provide an overabundance of water. This oversight could (and often does) result in the most serious problem of soil swell and foundation upheaval.

7B.6.12 Cost

Computing the cost for reliable, widespread, chemical stabilization is very difficult. This is generally because:

1. Many products are proprietary, and application procedure vary.
2. There is little history or cost data in the publications.
3. Treatment specifications and applications vary broadly.
4. There is no basic standard for acceptable performance. Standard Atteberg limit tests offer little value.

In fact, the only cost figures, which the author will stand behind, are those in the table on the next page for the chemical Soil Sta. Other data were acquired "second-hand," generally by word of mouth. Again, the labor costs used in placement costs should be computed as a relative rate (based on unskilled labor) at $7.00/hr and 1999 U.S. Dollars.

Method	Cost
Lime stabilization	
Mechanical mixing:	\$0.27/ft^2 per 6 in (30 cm), lift with 6% lime
Pressure injection:	\$0.23/ft^2 to 7 ft (2.1 m)
	\$0.15/ft^2 to 4 ft (1.2 m)
	\$0.13/ft^2, large Area
Chemical stabilization (pressure injection)	
Chemical A	\$2.17/ft^2 to 6 ft
Chemical B	\$0.20/ft^2 to 6 ft
Chemical C	\$15.50/ft^2

The following figures are for chemical stabilization using Soil Sta:

Area	New construction, \$/ft^2	Remedial, \$/ft^2
1000 ft^2	1.40	—
2000 ft^2	0.90	0.70
4000 to 8000 ft^2	0.80	0.70
8000+ ft^2	0.50	0.65

Lime stabilization accomplished by mechanical mixing is normally bid on the basis of \$ per yd^3. This cost was changed to \$ per ft^2 in an effort to present a better view of the comparison. The prices for Soil Sta concern:

1. The use of ⅛ gal of chemical per ft^2
2. Sufficient chemical to treat the soil to a depth of 6 ft (1.8 m)
3. Injection sites 5 ft (1.5 m) OC to a depth of 4–5 ft (1.2–1.5 m) for new construction. Injection holes on remedial applications are fewer in number (wider spaced) due to the specifics of the job.
4. Includes a 5 ft (1.5 m) apron around the footprint of the foundation

Only Soil Sta offered numbers specifically for remedial applications.

7B.7 CASE HISTORY

7B.7.1 Introduction

There are thousands of potential case histories. The following were selected principally because they offered something a little different.

7B.7.2 Florida Lake House

This problem involved extreme subsidence brought about principally when the water level in an adjacent lake was lowered several feet. The bearing soil consisted of a top layer of silty sand, a midlayer of decayed organics (peat), and a base of coral sand.

The two-story brick veneer dwelling suffered from differential foundation settlement reaching 6 in (15 cm) in magnitude. The objectives were to 1) underpin the perimeter beam to facilitate leveling (as well as provide future stability), 2) consolidate the peat stratum by pressure grouting to provide a solid base for the repaired structure, and 3) mudjack the entire foundation slab area to create a level or more nearly level structure. Figure 7B.7.1 depicts the repair process. First, excavations were made at strategic locations to provide the base for the spreadfootings (underpinning). Next, grout pipes were driven through the base of the excavations into the peat identified for consolidation. Steel-reinforced concrete was then placed in the excavations and allowed to cure. While the footing pads were curing, the entire slab was drilled for mudjacking and interior deep grouting. "Deep" grouting was then initiated, starting with the permanent grout pipe set through the footings and continuing to the interior.

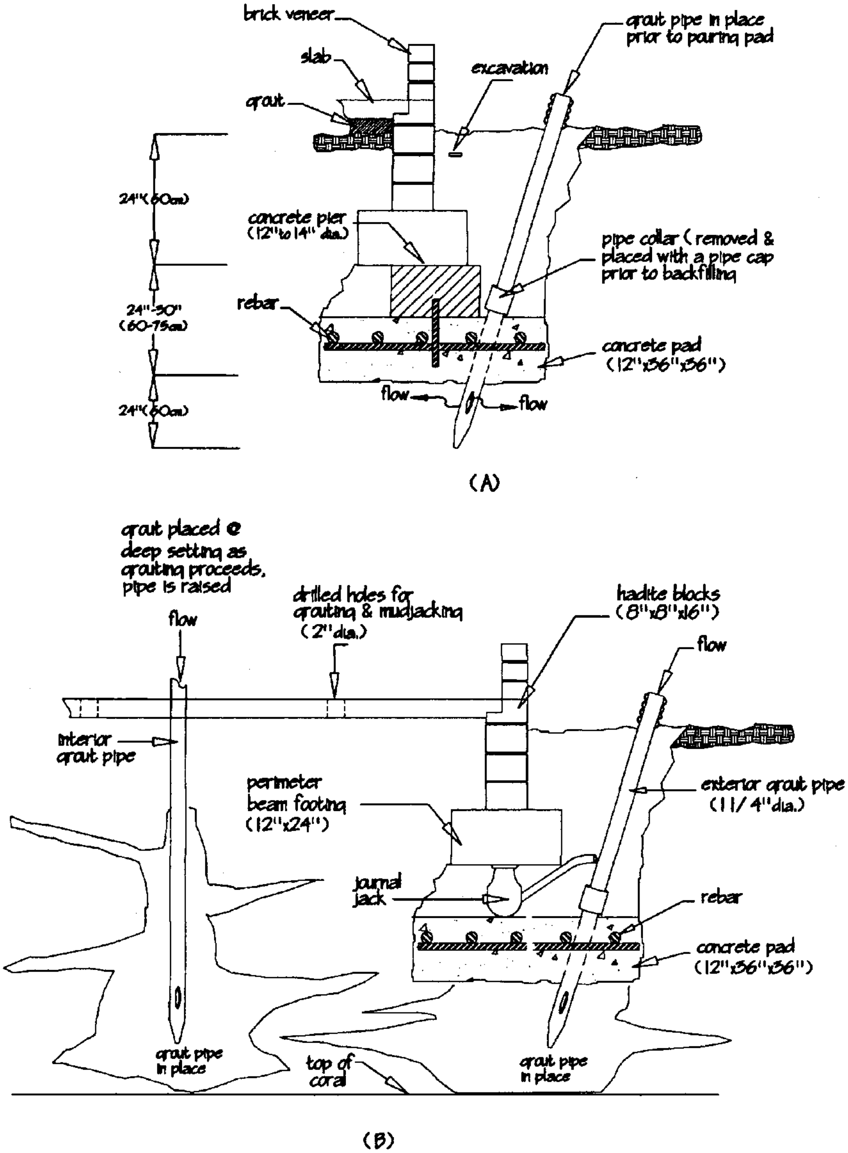


FIGURE 7B.7.1. Florida repair. (a) Grout pipe in place; (b) typical deep grouting procedure.

At each site, grouting was continued either to refusal, a break-through of grout to the surface, or the start of an unwanted raise. Interior grout pipes were removed after the grout process. The permanent, exterior pipes were cleared of grout and capped. After the grout had set, the top section of the perimeter pipes was removed and capped below grade. This procedure would permit regrouting at some future date should subsidence recur. Next, the perimeter beam was raised to desired grade and pinned by the installation of the poured concrete pier caps. The final step was to mudjack the entire foundation for final grading. Repairs were completely successful and have remained so since 1980. Prior to the work, the dwelling could be neither inhabited nor sold.

7B.7.3 Garages in London, England

The soil in the United Kingdom is not materially different from that found in parts of the United States. The plasticity indices classically run in the range of 40 to 50 and the problem clay (montmorillonite) content in the vicinity of 20 to 40%. The annual rainfall is not particularly high, only about 30 in (76 cm) of rain per year. The rain is well distributed over the year (150 days), with seldom more than 3 in (7.5 cm) during any one month. The Dallas Metroplex, by contrast, will have about 30 in (76 cm) of rain per year but perhaps 75 to 80% of the total will fall in less than 15 days. (The latter location experiences something like 90% run-off.) Further, London's high temperatures are generally in the 70^0 F (21 C) range, whereas the Metroplex highs exceed 105^0 F (40.5^0 C). Thus, London's climate produces a high, fairly consistent moisture content within the soil.

Often the soil moisture tends to persistently approach or exceed the plastic limit (PL), indicating little, if any, residual swell potential. [It is interesting to note that moisture contents taken from soil borings in close proximity to trees often show little, if any, reduction in percentage of water between the depth of approximately 1 m (3.3 ft) to perhaps 15 m (49 ft). Obviously, this suggests that, over the centuries, the soil has attained a level of unique moisture balance.]

Occasionally, a prolonged change in climate does come along that tends to temporarily disturb the balance, such as the drought of 1976. During that period, the soil moisture within the top 2 m (6.6 ft) or so was significantly reduced, reportedly causing severe and extensive problems of subsidence. Later, upon return of the normal moisture, the problems became even more severe due to soil swell and upheaval.

The London projects involved restoring the foundation of four banks of garages to the extent that the repairs could be expected to alleviate future distress for a period of at least 20 years. The repair procedure included the pressure injection of Soil Sta into the bearing soil beneath the foundations to a depth of 1.8 m (6 ft). The chemical was injected on the basis of ¼ gal/ft^2 (1 ml/cm^2) of the surface treated. Soil Sta was used to, hopefully, preclude the recurrence of the effects of drought conditions such as those of 1976, specifically the upheaval phase. Next, spreadfootings were installed beneath the load-bearing perimeter to permit mechanical raising and underpinning. Soil Sta was injected through the base of each footing excavation prior to pouring concrete. The chemical volume and purpose were the same as specified above. The final stage involved mudjacking to fill voids, raising and leveling the slab foundation, and filling any voids beneath the perimeter beam that resulted from the underpinning.

The jobs were considered successful. The procedure was substantially less expensive than the deep pilings or needle piers previously considered as a conventional approach. In addition, the foregoing methods cause less damage to the landscaping and are quicker and less involved to perform.

7B.7.4 Piers—Driven Steel Pipe: Another Example of Failure

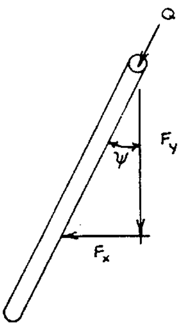
The field photographs in Figures 7B.4.9 and 7B4.10 show steel pipe placed by a hydraulic driver and pinned by bolts through the lift bracket. Figure 7B.5.9a shows the pipe slanted inward at over

30^0 (proving nonalignment). In this instance, if one assumes an axial load on each steel pile of 6000 lb (27 kN), the lateral vector (f_x) would be 3000 lb (13.5 kN). This force plus any lateral force created by the soil can be responsible for pile failure in lateral stress (refer to Section 7A.4.1 and Figure 7B.7.2). The lateral component of soil stress is also discussed in a section of the book by Prakash and Sharma[91] and in an article by G. G. Myerhoff.[74]

Figure 7B.5.9b shows the lift bracket not in contact with the beam. (The settlement of the pipe could be the result of soil dilatency, clay bearing failure, or ultimate failure in whatever material or object into which the pier tip is embedded.) The shiny spots on both pipes represent a prior attempt to adjust the pipes to reraise the beam. Figure 7B.4.9c represents the excavation of two minipiles at the corner of a foundation. Note the obvious bending and loss of contact between the perimeter beam and lift brackets. These piers are totally ineffective. Figure 7B.4.9 depicts only a few examples; however, this type of performance appears to accompany the driven steel minipipe procedures, at least when expansive soils are involved. Refer also to Section 7B.4.5.

Other conditions of load–pile/pier reaction exist. For example a vertical pile subjected to a inclined load Q at angle w is equivalent in behavior to a batter (rake) pile/pier inclined at an angle w and subject to vertical load Q. The simplest and preferred condition occurs when the pile/pier is vertical with the load applied concentrically.

It certainly seems safe to say that nonperformance represents the rule rather than the exception, at least within certain areas. Another problem, limited to slab foundations, has been the failure of contractors to follow the piling process with competent mudjacking of the slab. Since the slab is not designed to be a bridging member, voids, preexistent or created by raising the perimeter, encourage interior settlement of the floors. This must be circumvented by mudjacking. Proper mudjacking could, in fact, eliminate or minimize some of the other inherent deficiencies of the driven-pipe



1. **Assume a rake angle ψ of 30° and an axial load Q of 6000 lb (27 kN).**

$$\sin \psi = F_x/6000$$
$$F_x = (\sin 30°)(6000)$$
$$= (0.5)(6000) = 3000 \text{ lb } (13.3 \text{ kN})$$
$$F_y^2 = (6000)^2 - (3000)^2 = (36 \times 10^6) - (9 \times 10^6)$$
$$= 25 \times 10^6$$
$$F_y = (25 \times 10^6)^{1/2} = 5000 \text{ lb } (22.2 \text{ kN})$$

2. **Assume a rake angle ψ of 15° and the same load as in (1).**

$$F_x = (\sin 15°)(6000) = (0.259)(6000)$$
$$= 1554 \text{ lb } (6.9 \text{ kN})$$
$$F_y^2 = (36 \times 10^6) - (2.4 \times 10^6) = 33.6 \times 10^6$$
$$F_y = 5.8 \times 10^3 = 5800 \text{ lb } (25.8 \text{ kN})$$

3. **Assume a rake angle ψ of 5° and the same load as in (1) and (2).**

$$F_x = (\sin 5°)(6000) = (0.087)(6000)$$
$$= 522 \text{ lb } (2.3 \text{ kN})$$

The lateral component of the axial force Q reduces to zero as the rake angle ψ approaches 0°. This is an overly simplified example, but it serves to illustrate a point.

FIGURE 7B.7.2. Calculation of lateral force on raked steel minipiles.

process. Mudjacking alone will normally hold a raised slab foundation, provided proper maintenance procedures are instituted and followed (Section 7C).

7B.7.5 Lowering a Foundation

On rare occasions, it becomes desirable to both raise and lower sections of a foundation. Figure 7B.7.3 represents such a situation. As the elevations indicate (Figure 7B.7.3a), the south two-thirds of the foundation has settled significantly [up to 3 in (7.5 cm)], while the garage, north, northwest, and north-central beams have heaved [at least 2½ in (6.25 cm)]. (The elevations are considered accurate to the extent that relative grade positions are shown. The question lies in defining the true *differential* movement.) the floors were not in contact with the soil and the beams were originally underlaid with 12 in (0.3 m) void boxes. The void had been lost in many areas due to either beam settlement, filling in by soil, or some combination of both. The void area was to be restored by excavating beneath the beams as required, and/or raising the beam.

Conventional methods were used to raise the settled areas of the beam. Next, the existing piers were adjusted and extended to resupport the beam, instead of adding new drilled piers or spreadfootings. Refer to Figure 7B.7.3. (Spreadfooting pads were used as a base from which to raise the beam sections after the existing belled piers had been broken free from the beams.)

The heaved segments of the beam were lowered by excavating the soil beneath the affected length of the beam, supporting the beam on jacks (resting on spreadfooting pads as previously noted), removing any above-grade lengths of the pier, pouring appropriate additions to the existing pier at each location, reloading the adjusted piers by lowering the beam onto the “new” piers, and then finally removing the jacks.

At the completion of the project, grade elevation suggested that the maximum remaining vertical deflection in the foundation was less than ½ in (1.27 cm), whereas at the beginning the maximum deflection was 5½ in (14 cm). This particular job presents an interesting question. What is the merit of the lowering procedures? In undermining the approximate 250 linear feet (75 m) of beam, it was necessary to remove approximately 300 ft^2 (27 m^2) of floor and subflooring to gain access to the interior beams. This area, plus the perimeter beams, was then excavated. Refer to Figure 7B.7.3E. The cost of undermining, including floor replacement plus the attendant lowering operation, represented about one-half the entire foundation repair bill. This approach was necessary to give the customer what he wanted, but could some compromise have been more cost-effective? Perhaps not in this particular instance, since the home was vacant and on the market.

The author tends to generally disfavor lowering operations. Raising lower areas to meet the high ones is almost always the most practical solution, particularly when dealing with normal residential or light commercial construction. Nonetheless, this procedure does give the repair contractor another option.

7B.7.6 Apartment Building

This project involved a more complicated problem. The foundation distress was such that the masonry exterior walls were forced outward to the point where the second-story precast concrete floor slabs were pulled almost off their base. The foundation problem was addressed as a typical slab repair. The perimeter was underpinned (spreadfootings in this case) and the interior floor slab was mudjacked. Concurrently, the interior was crisscrossed with dywidag bars at the first-floor ceiling, extending through the drywall to the exterior. (Refer to Figure 7B.7.4.) The walls were plumbed as tension was applied by tightening the nuts on the dywidag bars against the steel beams or plates. All beams were later replaced with steel plates. A ceiling furr-down was used to conceal the bars on the inside. The exterior bars were cut flush with the nuts, plastered over, and painted to match the exterior walls. (Willard Smith & Associates, Dallas, performed the steel work.)

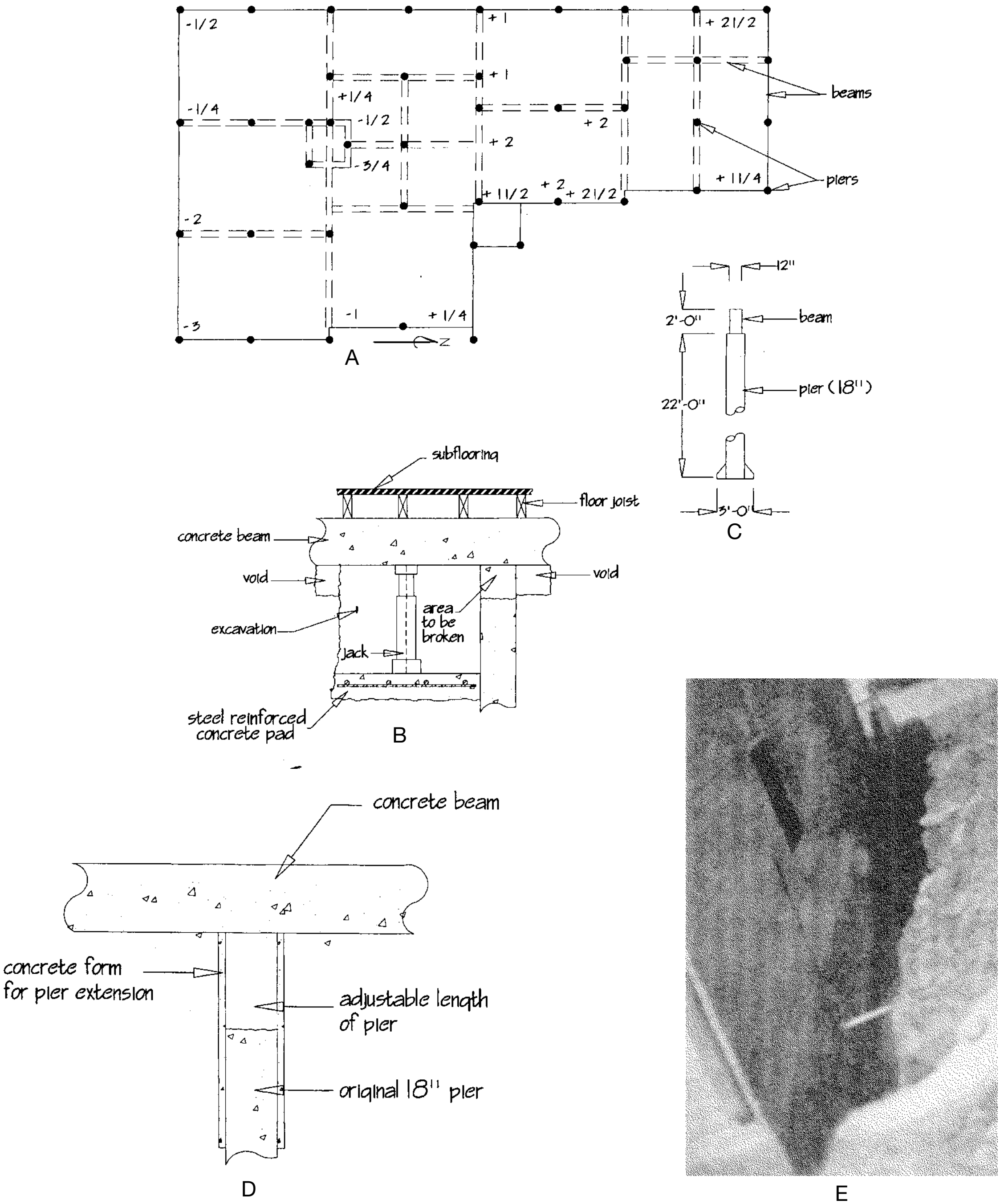
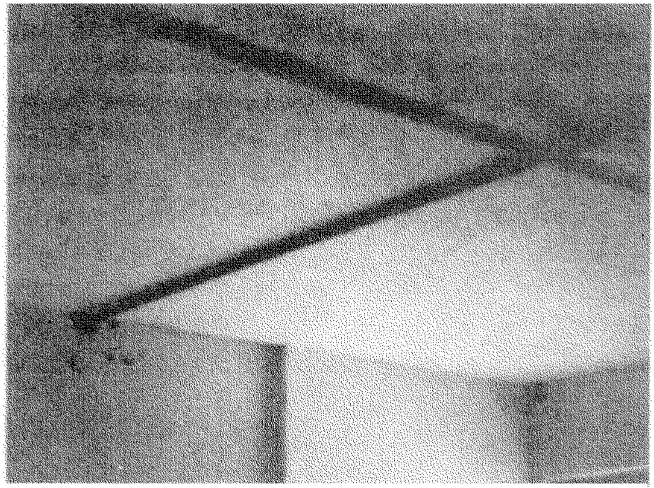
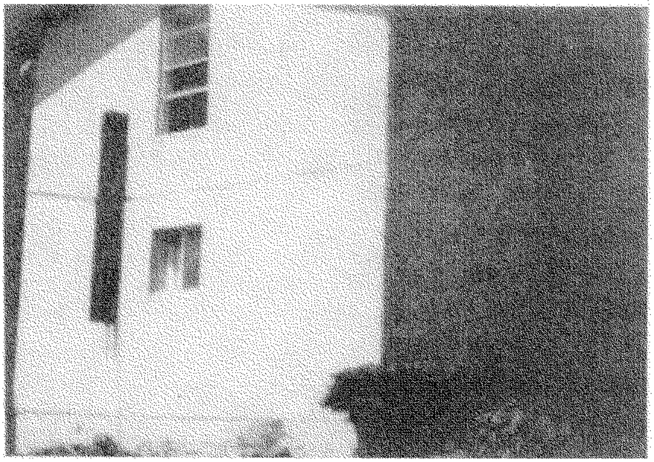


FIGURE 7B.7.3. Raising and lowering a foundation. (A) Foundation plan and elevation; (B) schematic drawing of mechanical setup for raising foundation; (C) pier detail (existing). (D) Preparing to pour new pier cap extension. (E) North wall of garage undermined. Note existence of original pier approximately beneath each window (refer to part A, floor plan.) The concrete pads to be used in the lowering operation appear at the bottom side of each pier. The pads for each corner pier are not evident.



(a)



(b)

FIGURE 7B.7.4. Apartment building repair. (a) Dywidag bars crisscross the interior space immediately below the first-floor ceiling; (b) steel beams distribute tension and allow walls to be pulled inward by tightening a nut on the dywidag threaded bar.

7B.7.7 Waco, Texas, Slab Foundation Not Properly Mudjacked

This represents another case where the slab foundation had been previously repaired and the effort was ineffective. Figure 7B.7.5 shows an actual photograph of the interior slab at the bath and an artist's concept of the predicament. In this example, the perimeter was underpinned using 12″ dia. (0.3 m) concrete piers (properly, it would seem, since the recurrent failure involved only the interior). The

(c)

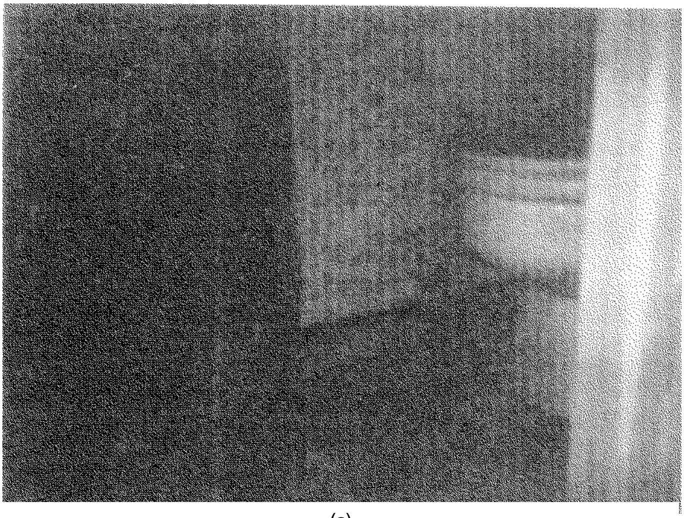
FIGURE 7B.7.4. (*continued*). Apartment building repair. (c) Steel beams are replaced with smaller plates once the wall has been plumbed. The excess bar is cut off flush with the nut and the assembly is stuccoed over and painted

interior was supposedly mudjacked. If mudjacking was, in fact, performed two facts seem apparent: 1) it was not thoroughly or properly completed and 2) there was no "telltale" evidence (patched drill holes) in the area. Restitution involved simply mudjacking the settled areas of the interior slab.

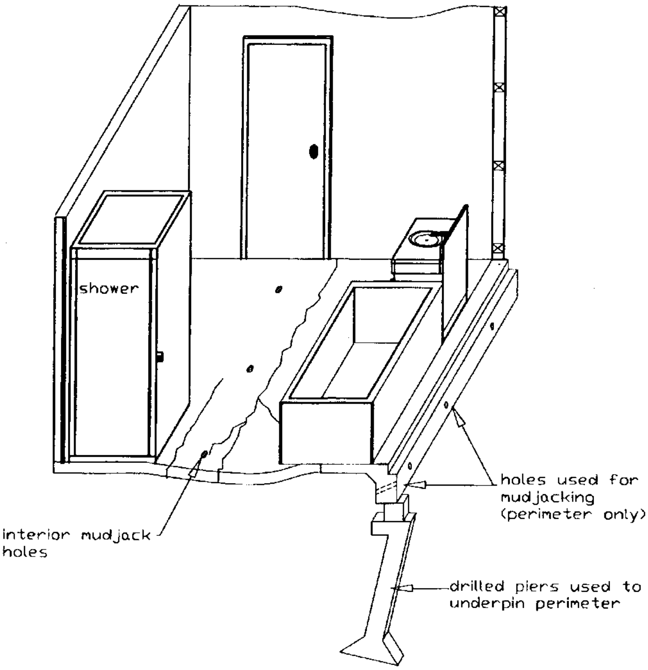
7B.7.8 Conclusions

Based on the foregoing, it becomes apparent that:

1. Foundation repairs tend to principally protect against recurrent settlement.
2. Recurrent upheaval is a concern, especially with slab foundations, unless thorough utility checks and proper maintenance are provided for.
3. Spreadfootings appear to be equal or, in some cases, perhaps superior to "deep piers" as a safeguard against resettlement[5].
4. "Deep piers" may, in some instances, be conducive to upheaval.[5] Although this observation is certainly true, the occurrence is not frequently documented. [The term "deep piers" refers to piers constructed to a minimum depth of about 10 ft (3 m).][17]
5. Upheaval accounts for more foundation failures (requiring repair) than settlement by a factor of about 2.3 to 1.0.
6. Moisture changes that influence the foundation occur within relatively shallow depths.
7. The effects of upheaval distress occur more rapidly and to a greater potential extent than do those of settlement.
8. The cause of foundation problems must be diagnosed and eliminated if recurrent distress is to be avoided.
9. Weather influences foundation behavior, both prior to and after construction.



(a)



(b)

FIGURE 7B.7.5. Slab failure due to failure to mudjack.. (a) Photograph depicting settlement of interior slab. In this instance, the condition was caused by incomplete mudjacking during initial reapirs. The interior slab was not properly mudjacked. (b) Artist's rendering of above condition.

7B.8 ESTIMATING

7B.8.1 Introduction

Estimating job costs forms the basis for any proposal. To establish these costs, the following factors should be considered:

1. Type of foundation: pier-and-beam, slab, posttension, slab-on-piers (are piers tied into perimeter beam?), etc. Are foundation plans and/or geotechnical data available?
2. Cause and extent of the problem. Settlement? Upheaval? Upheaval is more costly to repair. Cause *must* be identified and eliminated to preserve repairs.
3. Type, number, and placement of underpins. What access is available? (Access dictates which equipment can be used for drilling. Will concrete need to be broken out? (Refer to Sections 7B.4.2 and 7B.4.3.)
4. Amount of mudjacking required. This number is calculated on the basis of average raise multiplied by the area to be raised. Refer to Table 7B.2.1.
5. Is water available at job site?
6. Is an unobstructed work and set-up area sufficiently close to accommodate equipment? In most cases, the pump/mixing equipment for mudjacking should be within 150 ft (45 m) of the farthermost injection site. Without access, expensive alternatives must be considered. The latter could involve stage pumping (multiple pumps) or a very high water-to-solids ratio.
7. If mudjackaing involves extensive inside pumping, such as the case might be with a warehouse slab, other questions arise, such as: 1) Is the slab dowelled into the perimeter beam? If so, chances are the dowels must be cut to enable raising the slab at the perimeter beam. 2) Can mudjacking equipment be moved inside to facilitate access. Will exhaust fumes and dust become a hazard? 3) Are windows or doors available to route the pump hose to the work site? If the exterior walls are CMU (concrete masonry units), can access holes be created through the walls.
8. Is there sufficient work access in the crawl space? Is the area dry? A "no" answer to either questions can be costly. Refer to Section 7B.1.

Most of these issues are determined based on an inspection report. A typical example of such a report is provided in Figure 7B.8.1.

The inspection report depicted as Figure 7B.8.1 represents the heart of estimating. This report provides the readily available information and observations to permit an overview regarding the probable (or contributing) cause of the problem and at the same time provide detail of the issues affecting the appropriate repair. Each and every mark on this report is significant. For example, notice the comments regarding the East and West brick walls—"mtr.jts.reas.st." This translates to "mortar joints reasonably straight." This observation, taken into account with other factors such as the 2″ (5 cm) crown in the interior slab, helps identify upheaval as the culprit. The arrows, circles, and fractions appearing at the corners indicate the relative movement at those spots, if no fraction is given, the movement is less than ¼″ (0.6 cm). The X's indicate locations for piers. The X? represents a location that might require a pier but likely will respond to mudjacking.

Prior to repairs, the plumbing test was conducted. A substantial leak was detected in the master bath shower–commode area. The leak was repaired. From the inspection report, it can be learned, among other things, that the foundation repair to the slab will require the installation of 25 drilled piers (12″ or 0.3 m) plus 2 days mudjacking. Access is available in 22 locations to permit use of the regular tractor (Bobcat or Case) mounted drills. Three piers at the covered patio will need to be drilled by a limited-access rig. Water for the grout is available at the site and the maximum length of grout hose is less than 100 ft (30 m). It is necessary to break out concrete at three locations for pier access. Based on this, a typical bid might be $10,411.00. This is an exceptional cost, necessitated in general by upheaval. Refer to Sections 7B.4.2 and 7B.2.4. The basis for specifying the 2 days mud-

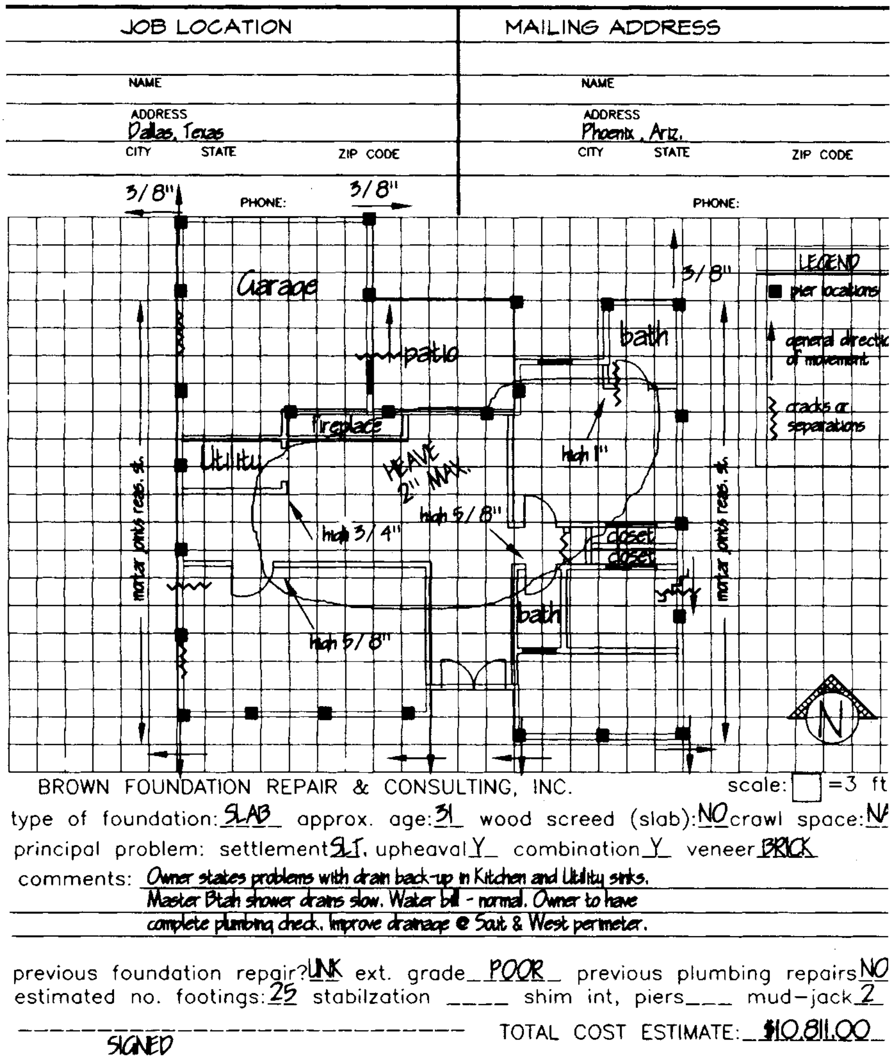


FIGURE 7B.8.1. Inspection report example.

jacking was skimmed over rather lightly. (The following sections and Section 7B.2 provide a better analysis of this.)

As an aside, it might be interesting to point out two concerns when utility leaks are detected beneath a slab foundation. If the leak is detected *prior* to initiation of foundation repair, it is often considered prudent to postpone repair procedure for several months to allow the bearing soil moisture content to reach some degree of equilibrium. This precaution might circumvent the need for the contractor to remudjack the interior slab at some future date. The second issue involves detection of the leak *after* repairs are under way. At this point, it is usually better to repair the leak and continue with the completion of work. This is particularly true if the perimeter has already been raised or is well along to being raised. The possible damage to an unsupported slab is more of a concern than the possible need to remudjack the slab at a later date. In either event, the problem is of more concern to the contractor than the consumer. The contractor's warranty protects the consumer.

7B.8.2 The Case For/Against Using Grade Elevations to Establish Amount of Raise Needed

Note the designated movement shown in Figure 7B.8.1. This gives guidelines concerning how raising can be permitted to restore the foundation to "as built" without creating undue "new" damage. The actual measurement of this differential movement is not normally reflected in grade elevations. The latter will reflect the contour of the foundation *at the time the measurements were made*. They do not necessarily reflect true movement. For grade elevations to be useful, there must be at least two sets taken over a period of time. The sets of elevations should use the same bench mark (assumed zero) and should probably be timed at least 6 months apart. Elevations taken at the time construction was first completed are always useful to compare with later readings. For example, refer to Figure 7B.8.2. The

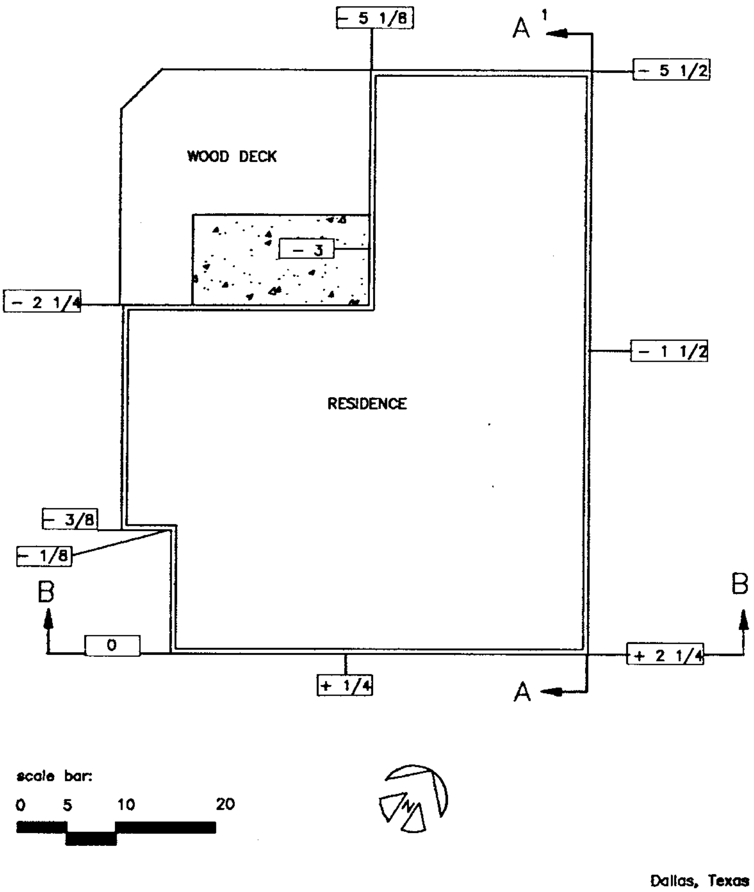


FIGURE 7B.8.2. Relative elevation survey.

grade elevations could suggest foundation movement in the range of 7⅜ in (19 cm). An inspection report found evidence of *differential* movement of less than 1 in (2.5 cm). If one attempted to raise this foundation 7 in (or even 4½″), devastating destruction would result. The elevations and the inspection report each serve as a useful tool to identify upheaval. This is very important because: 1) the cause of failure influences repair procedures and 2) the cause of foundation problems must be identified and *corrected* concurrent with any repairs; otherwise, the repairs cannot expected to be permanent. Due to the preponderance of upheaval in slab repairs, this importance is even more emphasized. More on this will be offered in Section 7B.8.3. For one reason or another some individuals, for self-serving reasons, have warped notions regarding the occurrence of upheaval, sometimes claiming that thousands of gallons of water are required to cause significant foundation heave. Others claim that "once the source of water is eliminated, the foundation will correct itself." Neither of these positions make sense. Consider Section 7B.8.3.

7B.8.3 Soil Swell (Upheaval) versus Moisture Changes

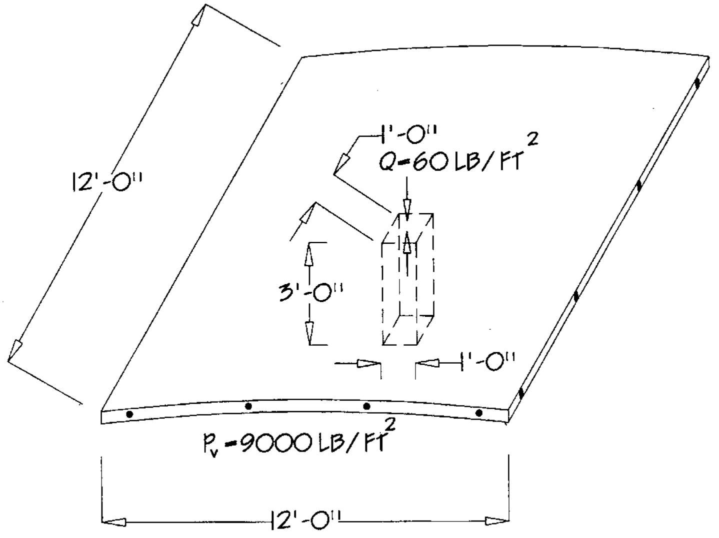
Some discussion has already been devoted to the issue of soil swell. Tucker and Davis presented data for a particular soil wherein a 4% increase in soil moisture was sufficient to cause a 1¼ in (3.2 cm) vertical rise in a type B, FHA slab foundation with a resulting potential swell force of 9000 lb/ft^2 (43,900 kg/m^2). This same study produced data that suggested a soil "active" depth of about 7 ft (2.1 m) (exterior to the foundation). However, these same data showed that over 85% of the total soil moisture change occurred in about the top 3 ft (0.9 m).[15,17]

What can all these seemingly unimportant factors tell about soil swell? First, refer to Figure 7B.8.3a. Here it is assumed that a cubic foot of soil is isolated from the surrounding soil. Assume an imaginary glass box 1 ft (0.3 m) square by 3 ft (0.9 m) deep filled with the soil described by Dr. Tucker, which existed at an initial moisture content of 20% with a final moisture content of 24%. The 9000 lb/ft^2 (43,900 kg/m^2) swell potential of the "confined" soil will tend to raise the lightly loaded slab [approximately 60 lb/ft^2 (290 kg/m^2)] over an area large enough to counter the upward thrust. In this case (9000 lb)/(60 lb/ft^2) = 150 ft^2 (13.5 m^2), or roughly an area of 12 ft (3.6 m) by 12 ft (3.6 m). This analysis admittedly takes certain liberties but is nonetheless technically valid. The *resistance* to heave would be materially influenced and enhanced by such factors as steel reinforcement, cross beams or load-bearing walls. However, the intent here is not to confuse but to illuminate. The resisting moment of a beam is a square function of its depth (steel reinforcement not considered); for example, under identical conditions, a wood beam 2 × 4 in (5 × 10 cm) on edge will support about 4 times the load it would if laid flat. When steel rebar within the concrete beam is considered, the moment of inertia (or stiffness) varies as the cube of the depth. In this case, doubling the depth of the beam would increase the rigidity by a factor of perhaps 8.[37,95]

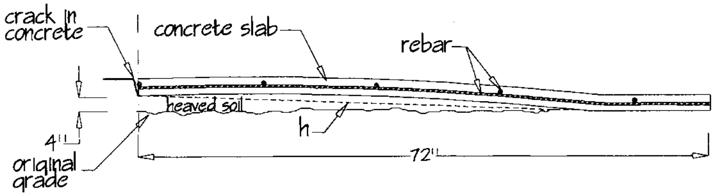
To pursue this train of thought, actually how much water is required to produce the 4% increase? Assume $W_S + W_w$ = 100 lb/ft^3 (1602 kg/m^3); W_S = 86 lb/ft^3 (1376 kg/m^3); initial moisture, W% = 16, W_w = 14 lb/ft^3 (224 kg/m^3); final moisture, W% = 20, W_w = 17 lb/ft^3 (272 kg/m^3); where W_w is weight of water, W_S is weight of soil, and W% = W_w/W_S. All values are based on a 1 ft^3 (16 kg/m^3) sample. Based on this, the added weight of water would be W_w = 17 – 14 = 3 lb/ft^3 (48 kg/m^3) or 0.36 gal (1.4 L) per cubic foot. Assume the constraints set forth by Figure 7B.8.4. In the case at hand, this would approximate only 1.2 gal (0.4 gal/ft^3 × 3 ft^3). Again, for simplicity, assume that the source for water had preexisted for 12 months. Then the daily input of water (and the amount required to produce the 4% increase) would be only 143 drops per day.[15–17]

For Example:

1.2 gal/12 months = 0.10 gal/month = 0.0033 gal/day
= 0.43 oz/day
= 14.3 mL/day
= 143 drops/day
= 0.10 drop/minute



(a)



$h^2 = 16\ in^2 + (72\ in)^2$

$h^2\ 16 + 5184 = 5200\ in^2$

$h = \sqrt{5200} = 72.11\ in$

stretch = (72.11 – 72.0)(2) = 0.22 in

or approximatel ¼'' over a 12 ft span
(this would involve rebar in heaved area)

In conventional slabs, the stress on rebar will not likely be uniform but generally localized to areas with concrete cracks.

In pt slabs, the stress will be more uniform to the cable with slab cracks more random.

(b)

FIGURE 7B.8.3. (a) Heave of concrete slab. (b) Rebar stretch versus heave. Assume that the heave depicted is 4 in (10 cm); the stretch in the rebar would approximate 0.22 in (5.6 mm). Again, this is an idealistic representation, but it serves illustrate an example.

Admittedly, this example isolates the cube of wetted soil, and in real life that could not occur. However, the relative quantities show a clear picture. Also, as a broader area is wetted, the potential soil heaved proportionately expands. In other words, if the wetted area expands laterally, the heaved area of the slab expands almost in direct proportion, although the magnitude of the heave could be lessened.

As the soil expands, what happens to the slab? First, assuming a conventional monolithic deformed bar slab, the steel is stressed to a distorted length that is not going to recover without some form of reverse stress. Along these lines, also consider Figure 7B.8.3b. If this slab is heaved by 4 in (10 cm), each steel rebar within the 12 ft (3.6 m) heaved area will be stretched by ¼ in (0.6 cm). Note as well that the elongation of the rebar will be anything but uniform. (With posttension slabs, where the cables are sleeved, this would not be the absolute case.) Even after the cause of the heave is alleviated, the domed area is not likely to return to a level (or near level) condition. In fact, experience dictates that the distressed area will not improve materially unless appropriate remedial actions are initiated.

Where can the water come from to cause the swell? This subject has been discussed to some extent in earlier paragraphs. Sewer leaks represent the most prevalent source. Figure 7B.8.5 depicts a sewer line with a separation. [The normal, minimal gravity fall in the sewer pipe is ⅛ in (0.3 cm) per linear foot—approximately 1 ft per 100 ft (0.3 m per 30 m).] Waste water directed into the sewer forms a vortex (turbulent flow), which creates centrifugal force tending to throw liquid from the pipe if any separation exists. Eventually, the flow settles down to the laminar regime, with the major velocity being down the pipe centerline. In laminar flow, the amount of water leaking from the pipe might be lessened. However, as shown in preceding paragraphs, very little water is required to cause a potentially serious threat.

Not all expansive soils swell when subjected to available water (see Figure 7B.6.2) If the existing moisture for these particular soils is above 24%, virtually all capacity for swell has been lost. Also, slab heave will not always appear to be as fairly uniform as depicted by Figure 7B.8.3a. The figure only illustrates a principle of force versus resistance and borders on an ideal condition. (For example, the wetted area is assumed to be at the surface. The affected area can be at the bottom of the plumbing ditch or even a foot or so below that. Also, the presence of porous fill and/or subsurface contact with foundation beams can influence the pattern of water flow. The latter would cause the

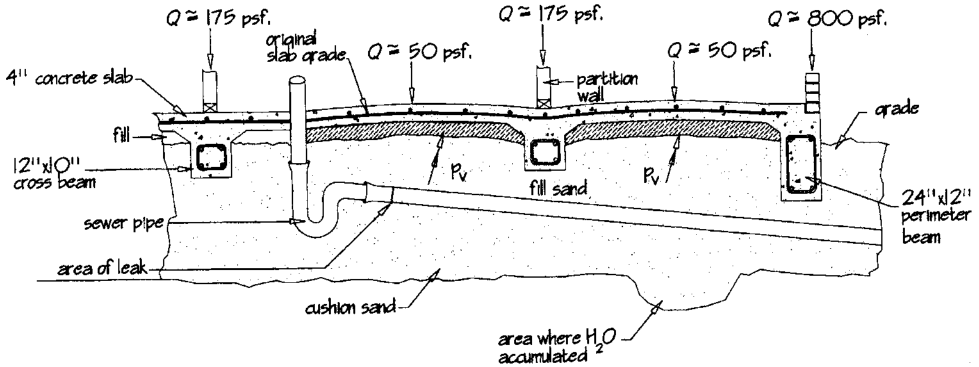


FIGURE 7B.8.4. Example of slab displacement due to upheaval resulting from soil swell. Note: the steel reinforcing and the cross-sectional cofiguration of the beams produce a resistance to stress far in excess of the structural loads indicated.

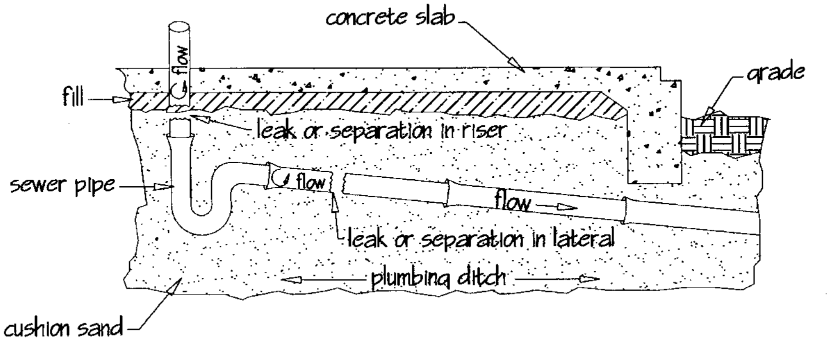


FIGURE 7B.8.5. Residential sewer.

heave to take an elongated rather than circular pattern. In many cases, particularly with older properties, proper mudjacking eliminates these flow channels.)

Realistically, the deformity of a concrete slab foundation would appear more as Figure 7B.8.4 suggests than as actually shown in the figure. The added structural load on the beam coupled with the increased resistance to deflection provided by the beam strength distorts the doming effect. In effect, the slab resembles a quilted surface except that the individual cells need not be of organized dimension. A topographical view of such a slab might resemble that in Figures 7B.8.6 or 7B.8.7.

Figure 7B.8.6 includes elevations with indications of minor settlement at the northwest, west of entry, and possibly the southeast corners. The major movement is center slab heave in the shaded area. Figures 7B.8.6 and 7B.8.7 also serve as foundation inspection field drawings intended to provide information sufficient for a repair estimate. The heaved area is generalized from observation of differential movement.

Section 7B.8.2 provides additional discussion regarding the difference between grade elevations and differential movement. Little, if anything, can be done to improve variations in grade elevation caused by initial construction. In fact, attempts to do so are likely to cause serious additional distress. Refer again to Figure 7B.8.2. It would be impossible to raise areas of this foundation to the extent the elevations suggest.

The foregoing will help provide the background information necessary to prepare a workable estimate. The appropriate repair can be balanced against the cause. Do all cases of foundation movement warrant repair? Who decides at what point repairs are feasible. Section 7B.8.4 will address these issues.

7B.8.4 How is the Need for Foundation Repair Established?

What extent of movement is sufficiently serious to warrant or demand repairs? There is no uniformly established rule. Several factors further complicate this issue: 1) Few, if any, residential foundations are constructed level; 2) a foundation being out of level does not normally render the property uninhabitable or unsafe; and 3) at least in part because of the foregoing, virtually all approaches to foundation repair (leveling) include some degree of compromise. The net goal is a foundation of "tolerable" appearance and behavior. The problem lies in the conceptual definition of "tolerable."

In attempting to establish a reasonable basis for defining the need and scope of foundation repair, the preferred approach is to provide the most acceptable appearance along with the most stable foundation, at the least possible cost. Determining the point of movement at which foundation repair is demanded is equally elusive.[65,73] Since most foundation repairs are performed as an aesthetic choice rather than a true structural necessity, the mental attitude of the property owner becomes a

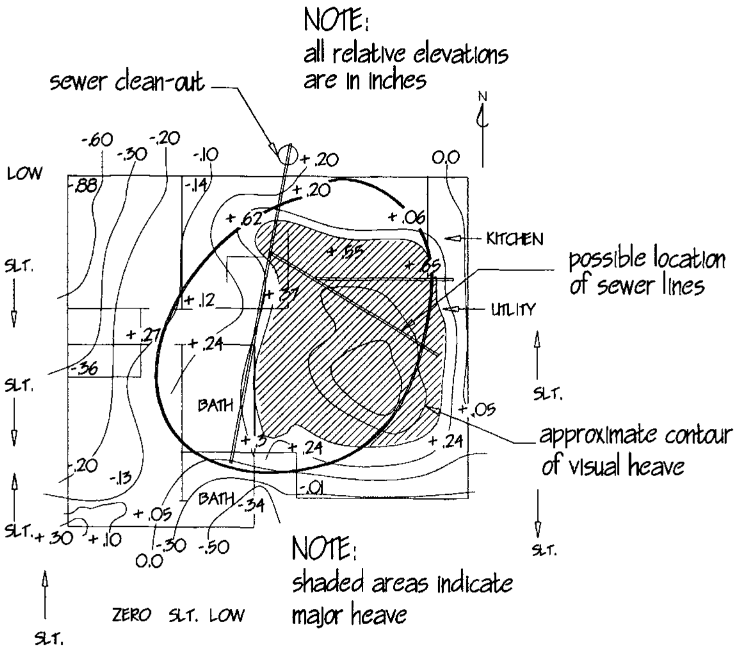


FIGURE 7B.8.6. Differential elevations in a slab foundation.

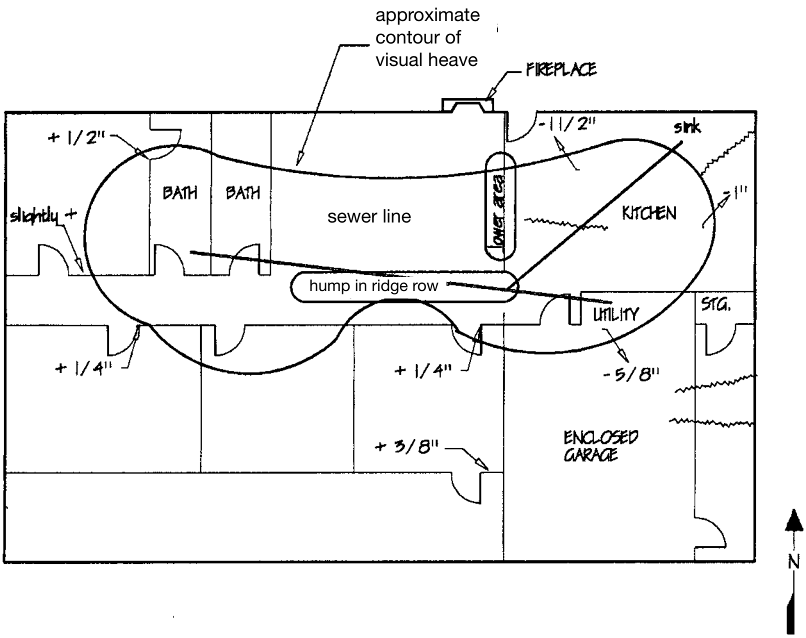
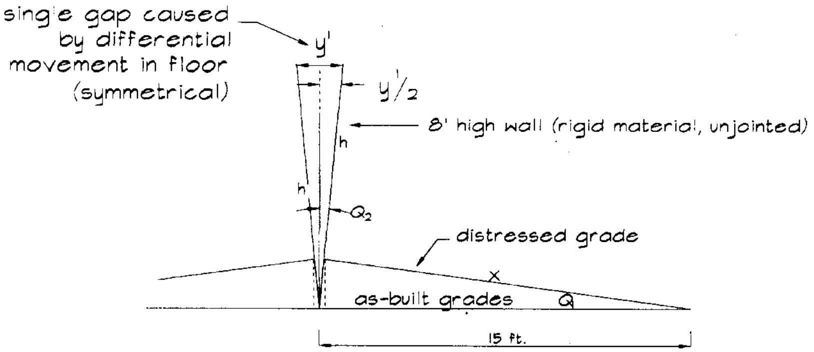


FIGURE 7B.8.7. Heave contour determined by field drawings.

prime factor. Other factors thatplay a part are property age and value, spendable income of the owner, peer pressure, cost of foundation repair versus continued cosmetic repair, is the movement ongoing, cause of distress (settlement versus upheaval), and the likelihood that proper maintenance could arrest the movement. Before any foundation repair is performed, the cause of the differential movement must be identified and subsequently corrected. Otherwise, recurrent problems are likely, regardless of the foundation repairs. A semitechnical approach for establishing "tolerable" and "intolerable" foundation movement is developed in following paragraphs.

Tensile strain for a typical mortared masonry wall is 0.0005 in/in (strain is determined by measuring deflection divided by distance). Thus, such a wall 8 ft (2.4 m) in height could resist movements up to about 1 in (2.5 cm) over 65 ft (20 m). On the other hand, a 1 in movement over 15 ft (4.6 m) could produce a single crack separation of about ¼ in (0.6 cm). Note the emphasis on "single crack" separation (See figure 7B.8.8.) Multiple cracks reduce separation width proportionately.

Lambe and Whitman present the relationship between strain and distance a bit differently.[65] Refer to Figure 7B.8.9. If *L* is replaced by *x* in *B* and by x/2 in *C*, the analysis is similar to that shown if Figure 7B.8.8. Consistent with this analogy, interior features such as door frames can tolerate movement on the order of 1 in (2.5 cm) over 12.5 ft (3.8 m). This is borderline. Increased movement usually causes the doors to be nonfunctional.



sin $Q_2 = y\ ½\ h$ and sin $Q_1 = y/x$
since $Q_1 = Q_2$

$$y^{1/2}h = y/x$$

$$\text{then } y' = \frac{2yh}{x}$$

solving for theoretical gap width:

let $x = 15'$ (180''); $y = 1''$ and $h = 8'$ (96'')

$$\text{then } y^{1/2} = \frac{1}{2}\left(\frac{2yh}{x}\right) \times \frac{(1'')(96'')}{180} = 0.55(y' = 106)$$

over 30' (360'') the gap width is y'

$$\text{then } y' = \frac{2yh}{x} = \frac{(2)(1'')(96'')}{360''} = 0.53 \text{ in}$$

FIGURE 7B.8.8. Theoretical single gap in 8 ft (2.4 m) wall.

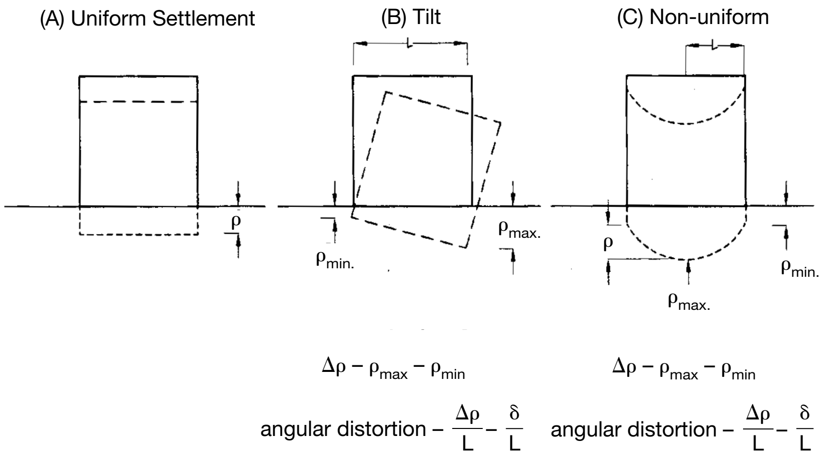


FIGURE 7B.8.9. Type of settlement.

This discussion brings into focus the problems inherent in defining "tolerable" deflection. Various conditions suggest different values. The distance over which the deflection occurs is the paramount concern, followed by the nature of the structure. An arbitrary value for acceptable movement recognized by some repair contractors is crack separation in excess of ¼ to ⅜ in (0.6 to 1 cm). These roughly equate to a differential movement of 1 in (2.5 cm) over 30 ft (10 m) or 25 ft (8 m), respectively. Normal construction often accepts a tolerance of as much as 1 in in 20 ft (6 m).[6,99B] This again emphasizes the difference between possible as-built grade variation and differential movement.

Kirby Meyer suggests the values expressed in Table 7B.8.1 as the criteria for foundation failure threshold.[73] In other words, movements that exceed the numbers indicated in the table suggest a failure condition, which should be considered as both serious and warranting remedial attention. Dov Kaminetzky approaches the situation from a slightly different perspective.[61] Table 7B.8.2 presents his classification of distress based on visible damage. He defines moderate damage as occurring at approximate crack widths of ⅛ to ½ in (3.2 to 12.7 mm). This is the point at which remedial action is suggested. Overall, the acceptable magnitude of differential movement is 0.3% (refer also to Section 7D).

A slightly different approach to a qualitative analysis for a slab-on-grade foundation is being considered by the Post Tension Institute. This directive is titled "Tentative Performance Standard Guidelines for Residential and Light Commercial Construction," as developed by the PTI of Phoenix, Arizona. This Bulletin addresses new construction (Type A) as well as existing structures (Type B). Concerns herein are limited to the latter. The following presents the "point" basis for foundation evaluation.

For Type B:

1. Foundation cracking
2. Sheetrock wall distress
3. Doors
4. Exterior cladding of brick, stone or stucco
5. Separation of materials
6. Foundation levelness

TABLE 7B.8.1 Recommended Foundation Failure Criteria for Selected Buildings, Maximum Allowable Values

Condition	Differential across two points, y/x*	Overall slope, % y/x*	Total vertical movement, in
Single or multifamily wood frame dwellings up to three stories			
Exterior masonry 8 ft high	1 inch/40 ft	0.21	
Exterior masonry > 8 ft high	1 inch/50 ft	0.166	
Exterior plaster 8 ft high	1 inch/40 ft		
Exterior nonmasonry	1 inch/16.6 ft	0.5	
Interior sheet rock walls 8 ft high	1 inch/30 ft	0.28	
Interior sheet rock walls >8 ft high	1 inch/40 ft		
Interior wood paneled walls	1 inch/76.7 ft	0.11	
Small-span structures with cross roof trusses	1 inch/30 ft		
All		(0.3) (1 in/30 ft)	4
Steel-frame building with metallic skin, no sensitive equipment, average range			
If isolated location with soft adjacent improvement and maximum drain	1 inch/16.7 ft	1.0	6
Concrete framed industrial building with CMU or masonry	1 inch/30 ft		
High exposure low-rise retail or office building with glass or architectural masonry walls	1 inch/50 ft	0.2	2 (but check entry transitions)

*x is lateral distance over which vertical deflection y occurs. In the case of overall slope, x would generally be the length or width of the foundation, as the case might be, and y the overall deflection.

Source: After Kirby T. Meyer, "Defining Foundation Failure".[73] See also Refs. 60, 61, 65, and 1.

Evaluation basis:

Item	Points
1. Foundation cracking (maximum points allowed: 7)	
a) Number of distinct cracks	
1–4	1
5 or more	2
b) Size of largest crack	
0″ to 1⁄16″	1
3⁄32″ to ¼″	3
5⁄16″ or greater	5

TABLE 7B.8.2 Classification of Visible Damage*

Extent of damage	Description of typical damage	Crack width, inches (mm)
	Hairline cracks of less than about 0.005 in (0.13 mm) are classified as negligible	
1. Very Light	Isolated light fracture in building. Cracks in exterior walls visible on close inspection.	1/64 to 1/32 (0.4 to 0.8)
2. Light	Light inside fracture visible on floors/partitions and on the exterior of buildings. Doors and windows may stick slightly.	1/32 to ⅛ (0.80 to 3.2)
3. Moderate	Moderate cracks are visible on the inside and on the exterior of buildings. Doors and windows stick. Utility pipes and glass may fracture. Water and air penetration through exterior.	⅛ to ½ (3.2 to 12.7) ½ to 1
4. Extensive	Severe cracks are visible on the inside and on the exterior of buildings. Windows and door frames are skewed and “locked in” noticeably. Walls lean or bulge noticeably. Some loss of bearing in beams. Cracks in utility pipes and glass brick, and water and air penetration through exterior.	(12.7 to 25.4)
5. Very extensive	Very extensive cracks are visible on the inside and on the exterior of buildings. Broken pipes and glass. Full loss of beam bearing; walls lean or bulge dangerously. Structure requires shoring. Danger of instability.	1 (25.4) or wider, but depends on number and location of cracks

*In evaluating the degree of damage, consideration must be given to its location in the building and structure (e.g., points of maximum stress). Crack width is only one aspect of damage, should not be used alone as a direct measure of damage, and refers to existing “old” cracks. New cracks must be monitored for possible increase in width. A criterion related to visible cracking is useful since tensile cracking is so often associated with settlement or movement damage. Therefore, it may be assumed that the state of visible cracking in a given material is associated with its limit of tensile strain.

Source: Dov Kaminetzky, “Rehabilitation and Renovation of Concrete Buildings.”[61]

Remarks: Corner wedge cracks are not considered distress cracks and score no points. Tight shrinkage cracks or hairline cracks too small for a business card to fit into also are not counted.

2. Sheetrock wall and cabinet distress (maximum points: 8)

a) Number of distinct wall cracks or wall/cabinet separations within the interior of the house	
1–5	1
6–10	2
11 or more	5
b) Size of largest sheetrock crack or separation	
0″ to ⅛″	1
³⁄₁₆″ to ⁵⁄₁₆″	3
⅜″ or greater	5

Remarks: Minimum crack length must be 6″.

3. Doors (maximum points allowed: 8)

a) Number of doors sticking or not latching properly	
1–3	1
4 or more	3
b) Any door totally inoperable	5

Remarks: Doors distress must be a function of foundation movement not due to material moisture changes.

4. Exterior cladding of brick or stone veneer or stucco (maximum points: 8)
   a) Number of distinct brick veneer cracks

1–4	1
5–10	2
11 or more	3

   b) Size of largest crack

0″ to ⅛″	1
¼″ to ½″	3
5/8″ or more	5

5. Separation of Materials (Maximum Points Allowed: 7)
   a) Number of locations where brick veneer and adjacent material are separated

1–4	1
5 or more	2

   b) Size of largest separation

0″ to ⅛″	1
$^3/_{16}$″ to $^5/_{16}$″	3
⅜″ or greater	5

Remarks: Normal caulk shrinkage and wood shrinkage do not count.

6. Foundation Levelness (Maximum Points Allowed: 5)
   a) Slope at any 8 ft line

0″ to ½″	0
$^7/_{16}$″ to ¾″	1
$^{13}/_{16}$″ to 1″	2
1$^1/_{16}$″ to 1¼″	3
more than 1¼″	5

Remarks: Elevation inconsistences representing a floor slope of greater than 1 ¼″ in 8 feet shall be considered unacceptable.

Conclusion. Any structure with a total score of 25 points or more is determined defective or "failed".

Example: Foundation and Superstructure Quantitative Criteria. A single-story, wood frame, brick veneer structure with a posttensioned slab-on-grade foundation. The property was built in approximately 1990. The evaluation follows. The system will be used to grade the condition of a structure. The structure will be considered a Type "B," since there were no prior slab elevation readings and the structure was existing. The system is quantitative and considers the structure to be defective or to have failed when a total score of 25 points is attained.

Evaluation:

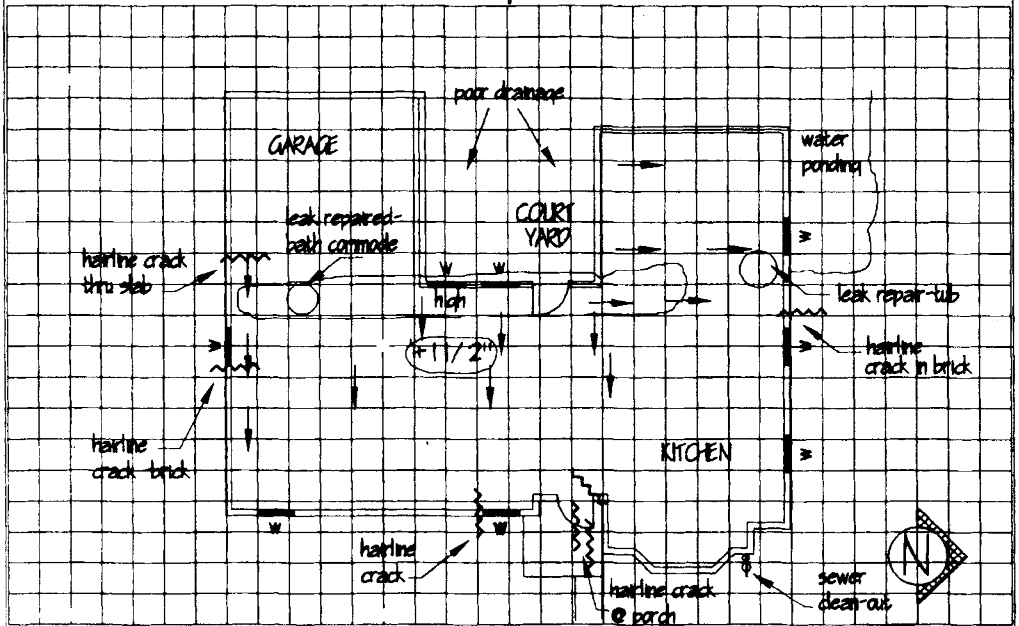
Item	Points
1. Foundation Cracking	
a) Number of Distinct Cracks	
3 Distinct Cracks	1
b) Size of Cracks	
$^3/_{32}$	3
2. Sheetrock Wall and Cabinet Distress	
a) Number of Distinct Cracks	
17 Distinct Cracks	3
b) Size of Largest Crack over 10″	
⅜″	5

3. Doors and Windows	
a) Number of Doors Sticking	
3 Doors and/or Windows Sticking	1
b) Any Door and/or Window Totally Inoperable	
3 Inoperable Doors/Windows	5
4. Exterior Brick Cracking	
a) Number of Distinct Brick Cracks	
6 Distinct Cracks	2
b) Size of Largest Crack	
1⁄32″	1
5. Separation of Materials	
a) Number of Locations	
4 Locations	1
b) Size of Largest Separation	
1″	5
6. Foundation Levelness	
a) Slope at any 8 ft line	
2¼″	5
Total Evaluation Points	**32**
The Worst Possible Score	48

From this analysis, this foundation has failed. This method is sometimes more tolerable than any of the foregoing, even though more observations are included in the evaluation. In order to equate the PTI evaluation to actual repair needs, this property was later inspected by a foundation contractor to provide a repair estimate. [The Engineer's evaluation of 1997 indicated a maximum grade differential of "plus or minus 2¼ in" (6 cm) or about 4½ in (12 cm).] The inspection drawing prepared by the contractor is given in Figure 7B.8.10. From this inspection, the magnitude of *differential foundation movement* is less than 2 in (10 cm). Note also that the principal problem is upheaval running essentially North to South between the two baths with prior leaks. As noted on other elevation exhibits, the extent of grade differentials is often not a reflection of differential foundation movement but a statement of original construction. Note that the maximum differential is stated to be about 1½ in (4 cm). In this case, foundation repairs are recommended subsequent to repair of the existing sewer leak.

Home Owners' Warranty (HOW) uses a still different approach. Their booklet describes "major structural" defects as those that meet two criteria: "(a) it must represent actual damage to the load-bearing portion of the home that affects its load-bearing function and (b) the damage must vitally affect or is imminently likely to produce a vital effect on the use of the home for residential purposes." In HOW Chapter 3, "Concrete," the following are cited as excessive cracks: foundation or basement walls—greater than ⅛ in (0.3 cm); basement floors—in excess of 3⁄16 in (0.45 cm); garage slabs—wider than ¼ in (0.6 cm); foundation slab floor—wider than ⅛ in (0.3 cm). Chapter 4, "Masonry," defines cracks in veneer as excessive when they exceed ⅜ in (0.9 cm) in width. Chapter 6, "Wood and Plastics," defines as intolerable floors that are out of level more than ¼ in (0.6 cm) over a linear distance of 32 in (0.8 m). General floor slope within any room should not exceed 1 in (2.5 cm) over 20 ft (6 m). On the other hand, HUD (at least some regional offices) often considers a property uninsurable if the floor is off level more than 11⁄16 in (1.7 cm) over 20 ft (6 m). This translates to 1 in (2.5 cm) over 30 ft (10 m) [18]. Both HOW and HUD offer the homeowner some measure of insurance. HUD insures homeowner mortgage loans and HOW provides them with insurance protection against major structural defects. Warranty Underwriters Insurance Co. and Home Buyers Warranty are examples of other organizations that offer home buyers some degree of protection in the form of insurance against major household defects such as foundation failure. Each entity offers somewhat different policies regarding insured losses.

JOB LOCATION	MAILING ADDRESS
NAME	NAME
ADDRESS DALLAS	ADDRESS
CITY STATE ZIP CODE	CITY STATE ZIP CODE
PHONE:	PHONE:



BROWN FOUNDATION REPAIR & CONSULTING, INC. scale: ☐ =3 ft.

type of foundation: SLAB approx. age: 7 wood screed (slab):__crawl space:__

principal problem: settlement__ upheaval Y combination__ veneer BRICK & WOOD

comments: November 1996 - repaired sewer leak. Major drainage problems @ rock courtyard & North wall. Correct drainage & reinspect in 2-3 months.

previous foundation repair? NO ext. grade POOR needs gutters previous plumbing repairs YES

estimated no. footings:__ stabilzation ____ shim int, piers___ mud-jack__

________________________ TOTAL COST ESTIMATE:________

SIGNED

FIGURE 7B.8.10. Inspection report.

Basic homeowners insurance policies generally cover foundation repair only in cases where foundation movements are caused by accidental discharge of water from a household plumbing system (sewer or supply).

If you have insurance and suspect a foundation problem, the best bet is to first contact a qualified engineer or foundation repair expert and follow their advice.

7B.8.5 Summary

General construction tolerance accepts grade differentials of up to 1″ over linear ft ("as built"). Most slab foundation designs are intended to tolerate differential movement(s) on the order of 1″ over 30 ft. To equate the two criteria to the accepted differential over the same distance, the comparison would be 1½″ over 30 linear ft for new construction as compared to 1″ over 30 linear ft for differential movement. The construction or "as built" tolerance is often concealed by framing. The differential movement(s) represents an applied stress that results in obvious structural distress. The 1″ over 30 ft is capable of producing single crack widths on the order of ¼″ to ⅜″ or a differential of about ⅛″ across a 3 ft door. Movement in excess of 1″/30 ft (0.33%) results in inoperable doors and generally unaccepted cracking. In the absence of distress there is normally no incentive for repair. This would be true even if slab elevations indicated a variation of up to 5″ over a slab length of 100 ft (with little or no evidence of structural distress).